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Mr. Ken Suter
Committee Chairman
OWRT—Tech Transfer
U.S. Department of the Interior
18th and "C" Street, N.W.
Washington, D.C. 20240
(202) 343-6883

Mr. Robert Madancy
Chief, Engineering Systems Division
OWRT—U.S.D.I.
18th and "C" Street, N.W.
Washington, D.C. 20240
(202) 343-6481

Dr. Edward H. Bryan
Advanced Environmental Research
and Technology
National Science Foundation
1800 "G" Street, N.W.
Washington, D.C. 20550
(202) 632-4356

Dr. K. Daniel Linstedt
WPCF—Reuse Committee Chairman
Department of Civil, Environmental and
Architectural Engineering
University of Colorado
Boulder, Colorado 80309
(303) 492-7007

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FORWARD

Wastewater reuse is one of several conceptual approaches to water conservation. It is based upon the realization that not all water users need water of the same quality. The idea of having a given quantity of water serve several different or repetitive functions before reintroduction to the environment is not new, but the rate at which it is increasing is.

Implementation of water reuse depends upon successfully satisfying constraints derived from considerations for environmental and human health effects, economics, process water quality, legal factors, and user acceptability.

The Symposium was the first week-long meeting devoted entirely to the renovation and reuse of wastewaters from municipal, industrial and agricultural sources. Unique to the Conference was the use of poster presentations for enhancing author-participant communication. Where quality photos were available, they are included at the end of several written manuscripts presented as posters.

These three volumes of Proceedings demonstrate that significant global interest exists in wastewater reuse; that recycling concepts are being applied to a broad range of functions; that application of water reuse may be highly situation-dependent; and that reuse applications are important solutions to conservation of water.

The objectives of the Symposium were to:

1. Provide program overviews
2. Present research results and practical experiences
3. Provide a forum for discussing the problems of reuse implementation
4. Encourage information exchange and the transfer of technology

These objectives were addressed and met by the 155 Symposium papers.

No attempt has been made to edit, reformat, or alter the material provided except for printing production requirements or where obvious errors or discrepancies have been detected. Any statements or views here presented are totally those of the speakers and are neither condoned nor rejected by the Symposium co-sponsors.



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MUNICIPAL REUSE

THE APPLICATION OF NON-POTABLE WATER
SYSTEMS TO RESIDENTIAL SERVICE

H. W. Stokes
General Manager and Chief Engineer
Las Virgenes Municipal Water District
4232 Las Virgenes Road
Calabasas, California 91302

M. E. Ford, Jr.
Vice President - Boyle Engineering Corp.
21 S. California St. P.O. Box 1748
Ventura, California 93001

CALIFORNIA BACKGROUND

Reclamation and reuse of domestic wastewater is growing throughout the world. The motivating factors include energy shortages, water shortages and the economy of conservation and reuse of a valuable resource. Reclamation and indirect reuse such as groundwater recharge or stream discharge is common. It has been estimated that over 25 percent of wastewater in California is disposed of on land or to inland rivers, creeks and lakes.¹ In states without access to the ocean this percentage is, no doubt, greater.

Irrigation and industrial uses currently account for the majority of direct reuse of reclaimed wastewater in California; those uses being on the order of six percent of all wastewater generated.¹ There are currently over 200 planned reclamation operations in the state.

Indeed, California law seems to encourage reuse of wastewater, for its statutes contain the following language:

"It is hereby declared that the primary interest of the people of the state in the conservation of all available water resources requires the maximum reuse of wastewater in the satisfaction of the requirements for beneficial uses of water."
- Water Code Chapter 6, Article 2

In furtherance of this policy, the governor recently (October 1977) established the Office of Water Recycling (OWR) as part of the State Water Resources Control Board to promote recycling and reclamation of water. The goal of OWR is to assist in implementing facilities to make an additional 493 cubic hectometres (400,000 a.f.) of reclaimed water available by 1982.

In addition, the California Department of Water Resources (DWR) has declared "the reuse of water to the maximum extent feasible" to be one of the elements of its water management policy.² In a recent talk before the State Board of Food and Agriculture the Director of Water Resources of California stated that "in 1980 the statewide net demands in water will exceed available dependable supplies by 2.5 million acre-feet." He presented Figures 1 and 2 which illustrate the anticipated water balance in 1980 and 2000 for the entire state and for the South Coastal Area, the location of the project discussed herein. It is interesting to note that of the statewide total of 600,000 A.F. of new reclamation in the year 2000, it is anticipated that 400,000 A.F. will occur in the South Coastal Area.

Thus, it appears the stage is set for a rapid expansion in reclamation and reuse of wastewater in Southern California.

LAS VIRGENES MUNICIPAL WATER DISTRICT

Las Virgenes Municipal Water District (LVMWD) was formed in 1958 under the Municipal Water District Act of 1911 of the State of California. LVMWD covers 78,500 acres in the western end of Los Angeles County. The District sits astride the rugged Santa Monica Mountains which reach a maximum elevation of 2,824 feet within three or four miles of the ocean. The major outlet to the ocean for the drainage of the interior valley is a water gap called Malibu Canyon. Figure 3 shows the general outline of LVMWD.

The population within LVMWD is on the order of 30,000 and is anticipated to exceed 65,000 by the year 2000. The majority of the population is located in residential developments in the valley, which is transected by U. S. Highway 101 (Ventura Freeway).

LVMWD is empowered to provide water and sewerage services to its citizens. Potable water is distributed throughout the area by an extensive water system. The potable water supply is from the State Water Project, delivered by the Metropolitan Water District of Southern California to LVMWD. The water is of relatively high quality for Southern California, containing approximately 350 mg/l total dissolved solids. Sewerage service is provided through a system of collector and trunk sewers which convey the wastewater to the 8 MGD Tapia Water Reclamation Facility (WRF) located at the head of Malibu Canyon.

Collector sewers are maintained and operated by County Sewer Maintenance Districts. Trunk sewers, Tapia WRF and reclaimed water facilities were constructed and are operated by LVMWD under a Joint Powers Agreement with Triunfo County Sanitation District (TCSD). TCSD provides sewerage service to the Malibu Canyon drainage area that lies outside LVMWD in Ventura County on the west and north. Approximately 12,000 persons reside in the rapidly expanding TCSD area.

LVMWD, through its citizens, Board of Directors and Staff, has a long-standing policy of supporting water reclamation and reuse. This policy stems from the general awareness that Southern California is a water deficient area and maximum use of its available water is essential. This is reflected in the high retail cost of the potable water supply (\$220 per acre foot), and the knowledge that large increases in the current wholesale rate of \$100 per acre foot by the Metropolitan Water District of Southern California can be expected when State Water Project power contracts are renegotiated in 1983. It is expected that retail water rates will increase to \$350 per acre foot by 1985.

Another contributing factor to the support of reclaimed water reuse is the opposition that exists to the discharge of reclaimed water from Tapia WRF to Malibu Creek. Inaccurate, colorful statements by a vocal minority have, of course, been picked up and quoted liberally by the press. District efforts to develop a viable reclamation, reuse and surplus reclaimed water disposal program have been repeatedly delayed. However, persistence on the part of the LVMWD Board and concerned citizens has finally resulted in the program moving forward. Part of this program is a non-potable water system for residential service.

RECLAIMED WATER DEFINED

It is necessary, at this point, to define two terms used frequently herein. The first is "reclaimed water." Reclaimed water, by practice in LVMWD is a domestic wastewater which has received secondary treatment by the activated sludge process, resulting in a nitrified effluent that meets the following key quality parameters:

<u>Parameter</u>	<u>Average Annual Values</u>
Coliform	less than 2MPN/100 ml
BOD	4 mg/l
Suspended Solids	4 mg/l
Turbidity	<2
pH Range	6.9 to 7.4
TDS	900-1065 mg/l

A complete listing of various water standards and the Tapia effluent quality is shown in Figure 4.

While at most times the water is in parameter compliance with the most stringent California standards for beneficial use of reclaimed water, namely those for spray irrigation of food crops and non-restricted recreational impoundments (body-contact), it is not coagulated and filtered, therefore, not in technical compliance.

NON-POTABLE WATER DEFINED

The next term is "non-potable water." Non-potable water, as used herein, means a water which at all times meets or exceeds the "body-contact" standards of the California Administrative Code, Title 22, which establishes reclamation criteria for various uses. This standard states that the water shall be at all times, an adequately disinfected, oxidized, coagulated, clarified, filtered wastewater. The wastewater is considered adequately disinfected if at some location in the treatment process the median number of coliform organisms does not exceed 2.2 per 100 ml, and the number of coliform organisms does not exceed 23 per 100 ml in more than one sample within any 30-day period. Filtered wastewater means an oxidized, coagulated, clarified wastewater which has been passed through natural undisturbed soils or filter media so that the turbidity does not exceed an average operating turbidity of two turbidity units, and does not exceed 5 turbidity units more than five percent of the time during any 24-hour period.

As noted earlier, although the current physical parameters of the Tapia "reclaimed water" meet this standard, LVMWD is currently designing a filtration plant that will provide the necessary strict compliance with the "body-contact" standards to meet the District definition for "non-potable" water.

The only remaining problem with the water is not health related, but the TDS. The District has had an ongoing program of infiltration management, however, a couple of recent earthquakes and a wet winter or two have compounded the problem. Currently, an accelerated review and rehabilitation program is underway. The problem is not volume of infiltration, but the high TDS of what passes for ground water in the District, ranging between 2000 and 5000 ppm.

RECLAIMED WATER SYSTEM

LVMWD has been delivering reclaimed water for agricultural and institutional landscape irrigation from its system shown on Figure 3 since 1971. The basic system consists of a pumping station at Tapia WRF, 5 miles of 16-inch and 18-inch pipeline in Las Virgenes Road and a 45,000 M³ (12 M.G.) storage reservoir adjacent to the LVMWD Headquarters. This system is currently capable of delivering up to 6 MGD. Lands adjacent to the system in Las Virgenes Valley, have until recently, provided sufficient acreage for crop and landscape irrigation or waste spraying to allow LVMWD to refrain from summertime discharge to Malibu Creek, a requirement of its NPDES permit. This situation changed when the California Department of Parks and Recreation acquired a large part of the Las Virgenes Valley and designated that the land be kept in its natural state, unirrigated. Recently (March 1978) LVMWD prevailed upon the State Water Resources Control Board to direct the California Regional Water Quality Control Board (Los Angeles Region) to amend their NPDES permit to allow a one-year, extensively monitored discharge of reclaimed water to Malibu Creek, while concurrently pursuing a program to develop additional beneficial reuse opportunities for the reclaimed water. In addition, the new filters at Tapia WRF will also help the effluent meet the non-potable standards required for permanent Creek discharge.

NON-POTABLE WATER SYSTEM CONCEPT

The dilemma with which LVMWD is faced is how to develop beneficial reuse opportunities in a mountainous area containing minimal agricultural and industrial development. This has led the District to one of the remaining markets capable of absorbing the non-potable water supply during a large part of the year, the residential homeowner. Table A indicates residential irrigation is on the order of 40% of residential water use, while toilet flushing adds another 20% plus, a substantial market indeed.

TABLE A

RESIDENTIAL WATER USE IN CALIFORNIA, 1972

EXTERIOR

A. Irrigation of lawns, shrubs and home gardens	39.6%
B. Other uses--car washing, swimming pools, cleaning driveways, sidewalks, streets, etc.	4.4%

INTERIOR

A. Toilet flushing	23.5%
B. Bath	17.9%
C. Laundry	7.9%
D. Kitchen	4.2%
E. Cooking	2.2%

Source: State of California, Department of Water Resources, Bulletin No. 198, "Water Conservation in California," May 1976

The District's preliminary proposals for the non-potable water system included both residential landscape irrigation and toilet flushing;

however, toilet flushing has been abandoned for the time being because of the adamant opposition of the health authorities. (We find it peculiar that these authorities consider that people might drink water from their toilets.) Notwithstanding, the residential landscape irrigation market for non-potable water is well worth pursuing in its own right.

During 1977 and 1978 LVMWD and Boyle Engineering developed the concepts for a dual water supply consisting of separate water systems for potable and non-potable water, with appropriate safeguards to minimize accidental ingestion of the non-potable supply by the user. When considering "appropriate safeguards" it must be borne in mind that the non-potable water supply will meet so-called "body-contact" standards of the State of California, as promulgated by the Department of Health and described earlier. In other words, swimming in water of this quality, or irrigating lettuce, is permitted.

The concepts that have been developed for LVMWD's non-potable water system are broadly described as follows:

The non-potable water system will be operated at a lower pressure than the potable water system. (Should a cross-connection occur, flow will be from the potable to the non-potable system.)

All non-potable facilities will be easily identified and differentiated from the potable facilities. (This will be accomplished by special marking and/or use of different materials.)

The installation and operation of non-potable facilities will be closely monitored by LVMWD, who will keep accurate records of the form and location of each on-site system.

LVMWD has adopted the necessary Resolution to provide for District control of the operation of the non-potable system through a permit system, and for enforcement of the rules and regulations relative to the use of the non-potable supply.

LVMWD will develop and implement an on-going education program for users of the non-potable system.

These concepts and the Resolution have been given qualified endorsement by the health authorities, and are being refined and expanded into detailed working documents which will govern the District's program for providing non-potable water service.

The concept of piping reclaimed water to the point of use has the advantage, when compared to ground water recharge and/or river or lake discharge, of sidestepping the residual organic question. By this means, virtually all beneficial uses, short of drinking, are within reach of present day knowledge and technology.

INITIAL APPLICATION OF A NON-POTABLE WATER SYSTEM

The initial opportunity for a dual water supply system occurred in 1978, and resulted from an expansion of the District's reclaimed water system. As a result of P.L. 92-500, Step 1, Area Wide Planning, prepared by the District's Consultant, Boyle Engineering Corporation, Clean Water grants for design and construction of a delivery system

to the Calabasas Park Golf Course were authorized. This system is now under construction and is shown on Figure 3.

This new system extension made it possible to offer a non-potable water supply to the developer of two residential housing tracts adjacent to the golf course. With the full cooperation of the developer LVMWD presented a discrete project for a dual water system to the health authorities. The initial proposal was for non-potable residential landscape irrigation of the front and rear yard, as shown in Figure 5, as well as a reclaimed system for greenbelt and contiguous slope irrigation. Figure 6 shows the original concept of the onsite valve termination.

After many discussions with the health authorities, the project was modified to include: timers that restrict the delivery of non-potable water to the homeowner to late evening and early morning hours; and the elimination of irrigation of rear and side yards with non-potable water. While contiguous greenbelts and slopes can be irrigated during the day by homeowner association employees, front yard landscape irrigation can occur only under controlled circumstances during late night or early morning hours. Side and rear yards under control of the homeowner must be irrigated with water from the domestic system. This compromise resulted in health authority support and subsequent adoption of "Reclaimed Water Requirements for Las Virgenes Municipal Water District's Non-Potable Water System" by the Los Angeles Regional Water Quality Control Board. This document provides the regulatory agency authorization and control over the non-potable water system.

Figure 7 shows the compromise onsite system layout for a typical single-family house being supplied with non-potable water. Figure 8 shows the compromise concept for valve termination, including the dual timers which provide the necessary District control of irrigation periods.

Figure 9 is a layout of a typical dual water distribution system in a subdivision receiving the non-potable residential irrigation supply. This concept has not changed from the original proposal. The pressure on the non-potable system will be regulated, and maintained, at a set differential below that of the potable water system, by a pressure reducing station, as shown schematically in Figure 10. The manufacturer of the differential pressure reducing valve has already developed a modified differential pilot control valve that has an open air separation between the potable and non-potable water ways in the valve. It is anticipated that this will provide ample (and acceptable) cross-connection protection in case of a failure of one of the diaphragms in a pilot valve. The pressure reducing facility will also provide a master meter (not shown) to measure the non-potable water delivered to the tract. The non-potable waterline in the street right of way will be asbestos cement pipe installed on the outside of the sidewalk. A magnetic red plastic tape marker with the words "Reclaimed Water" will be placed atop all pipes.

Non-potable water service and onsite lines will be plastic. The box covering the curb stop, and a standard pressure reducing valve if required, will also be made of plastic. This is in contrast to the concrete meter box used for potable water service. All buried onsite non-potable lines will be marked with plastic tape, the same as the distribution facilities. No hose bibs will be permitted on the non-potable system, and all sprinklers will be stationary. All hose bib outlets on the property will be from the potable system; thus, land-

landscape irrigation can continue by hose connections should there be an interruption in the non-potable supply, or should the property owner not desire to irrigate during night time or early morning hours.

It is anticipated that non-potable water for residential landscape irrigation water will initially be delivered on a flat rate basis, with no meters on individual services. As stated previously, should a cross-connection occur, accidentally or otherwise, water from the potable water system will flow to the lower pressure non-potable system. Thus, an unusual increase in potable water consumption would occur, with a marked increase in the bi-monthly water bill, thereby automatically indicating that the premises should be immediately checked for a cross-connection.

Reclaimed water system customers on the other hand, will probably be metered, and also will usually have backflow prevention devices at the meter.

SYSTEM OPERATION

The guidelines for operating the non-potable water facilities are contained in a Resolution adopted by the Board of Directors of Las Virgenes Municipal Water District. A synopsis of pertinent portions of the Resolution may be helpful in understanding the close control the District envisions maintaining over the non-potable system.

Offsite Reclaimed Water Distribution System

Operation and surveillance of all of the District's offsite reclaimed water distribution system including, but not limited to, reclaimed water pipelines, reservoirs, pumping stations, fire hydrants, manholes, valves, connections, supply interties, treatment facilities, and other appurtenances and property up to and including the District's meter serving the customer, shall be under the management and control of the District. No persons except authorized employees of the District shall have any right to enter upon, inspect, operate, adjust, change, alter, move, or relocate any portion of the foregoing, or any of the District's property.

Onsite Reclaimed Water Systems

The operation and surveillance of all onsite reclaimed water system facilities; including but not limited to landscape irrigation systems, agricultural irrigation systems, systems utilized in relation to use of reclaimed water for industrial process or construction purposes, or recreational impoundment systems using the District's reclaimed water; shall be under the management of an "Onsite Reclaimed Water Supervisor" designated by the applicant, owner, or customer and approved by the District. The District may, from time to time, require that an "Onsite Reclaimed Water Supervisor" obtain instruction in the use of reclaimed water, such instruction being provided by, or approved by, the District. However, the Manager or authorized representatives of the District shall monitor and inspect the entire reclaimed water system, including onsite and offsite facilities, and for these purposes shall have the right to enter upon the customer's premises during reasonable hours.

The applicant, owner, or customer shall have the following responsibilities in relation to operation of onsite facilities:

1. To make sure that all operating personnel are trained and familiarized with the use of reclaimed water.
2. To furnish their operating personnel with maintenance instructions, controller charts, and record drawings to ensure proper operation in accordance with the onsite facilities design and these Rules and Regulations
3. To prepare and submit to the District one (1) set of record drawings.
4. To notify the District of any and all updates or proposed changes, modifications, or additions to the onsite facilities, which changes must first be approved by the District, and which must be designed and constructed according to the requirements, conditions, and standards set forth in the District's "Standard Specifications for Construction of Reclaimed Water Facilities" and set forth in these Rules and Regulations. In accordance with the above referenced requirements, conditions, and standards; changes must be submitted to the District for plan check and approval prior to construction. The construction shall be inspected by the District, and revised record drawings and controller charts shall be approved by the District. The District may, if it deems such to be in the best interest of the District, waive or modify any of the foregoing.
5. To ensure that the reclaimed water facilities remain in accordance with these Rules and Regulations including the District's "Standard Specifications for the Construction of Reclaimed Water Facilities." For example, but not by way of limitation, as stated in the design criteria section of the above referenced specifications:
 - a. Cross-connections between potable water facilities and reclaimed water facilities are forbidden.
 - b. Hose bibs on reclaimed water facilities are forbidden.
 - c. Drinking fountains shall be protected from the spray of reclaimed water.
6. To operate and control the system in such a manner as to prevent direct human consumption of reclaimed water, and to control and limit runoff. The applicant, owner or customer shall be responsible for any and all subsequent uses of the reclaimed water. Operation and control measures to be utilized in this regard shall include:
 - a. Onsite reclaimed water facilities shall be operated to prevent or minimize discharge onto areas not under control of the customer. Part circle sprinklers shall be used adjacent to sidewalks, roadways, and property lines to confine the discharge from sprinklers to the designated areas.
 - b. The operation of the onsite reclaimed water facilities shall be during the periods of minimal use of the service area. Consideration shall be given to allow a maximum dry-out time before the designated areas will be used by the public.

- c. Reclaimed water shall be applied at a rate that does not exceed the infiltration rate of the soil. Where varying soil types are present, the design and operation of the reclaimed water facilities shall be compatible with the lowest infiltration rate present.
- d. When the application rate is seen to exceed the infiltration rate of the soil, automatic sprinkler systems shall be reprogrammed to prevent or minimize the ponding and runoff of reclaimed water. The sprinkler shall not be allowed to operate for a time longer than the landscape's water requirement. If runoff occurs before the landscape's water requirements are met, the automatic controls shall be reprogrammed with additional shorter water cycles to meet the requirements. This requirement is intended to control and limit runoff.
- e. To report to the District any and all failures in their system that cause an unauthorized discharge of reclaimed water.

Onsite Non-Potable Water Systems

The above requirements for onsite reclaimed water distribution systems include the basic requirements for the non-potable system. The homeowner is, of course, the onsite reclaimed water supervisor for his non-potable system, and will be expected to understand and live up to the requirements of the Resolution. The major differences between the reclaimed water requirements and non-potable water requirements are: the use of the District-controlled timer to limit the hours the homeowner can irrigate his front lawn; the prohibition against side and backyard irrigation with non-potable water, which is, hopefully, only temporary, while the physical administrative controls go through a shakedown period; and each homeowner, or private person using non-potable water will be required to obtain a permit to receive the water, and to use it only for irrigation purposes. Violations of permit conditions will result in various punitive actions against the owner, depending on the severity of the violation.

A non-potable system user education program has not yet been developed; however, one of the conditions imposed by the Los Angeles Regional Water Quality Control Board requires their approval of the ongoing education program for users of the non-potable system, prior to delivery of non-potable water.

CONCLUSIONS

The granting of permission by the California regulatory agencies to provide non-potable water service for residential front yard landscape irrigation with water derived from treated, domestic wastewater is an important step forward in meeting reclamation and reuse goals of the State of California, as well as LVMWD. It is believed that, in time, permission will be extended for unrestricted residential landscape irrigation. Furthermore, it is anticipated that, in time, the opposition of the health authorities to the use of this water for toilet flushing will mellow. LVMWD intends to continue its efforts to provide safe, non-potable water through a secondary water distribution system to enlightened and educated consumers. In this manner the District will be living up to the spirit of California law, which requires the maximum reuse of wastewater, and will be taking advantage of the

of the beneficial economics of a socially-acceptable program, resulting in lower short and long-term total cost to the customer for water and sewer service.

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3. "Dust Bin of History" by Ronald B. Robie, Director of Water Resources, Talk presented to State Board of Food and Agriculture, Sacramento, California, February 1, 1979.

STATEWIDE WATER REQUIREMENTS AND SUPPLIES

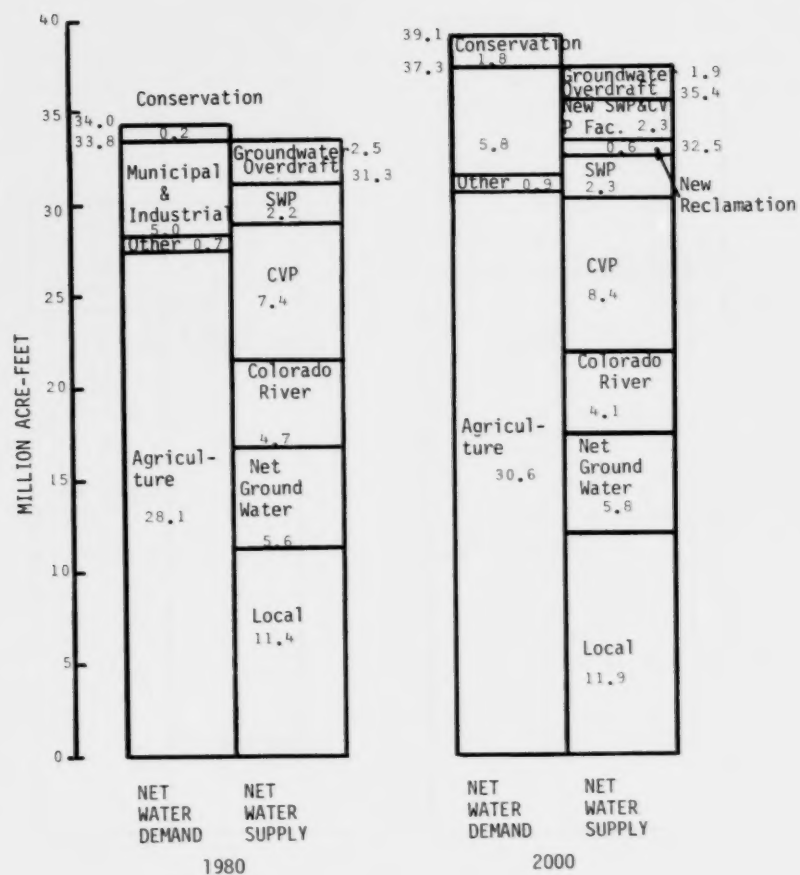


FIGURE 1

SOUTH COASTAL AREA WATER REQUIREMENTS AND SUPPLIES

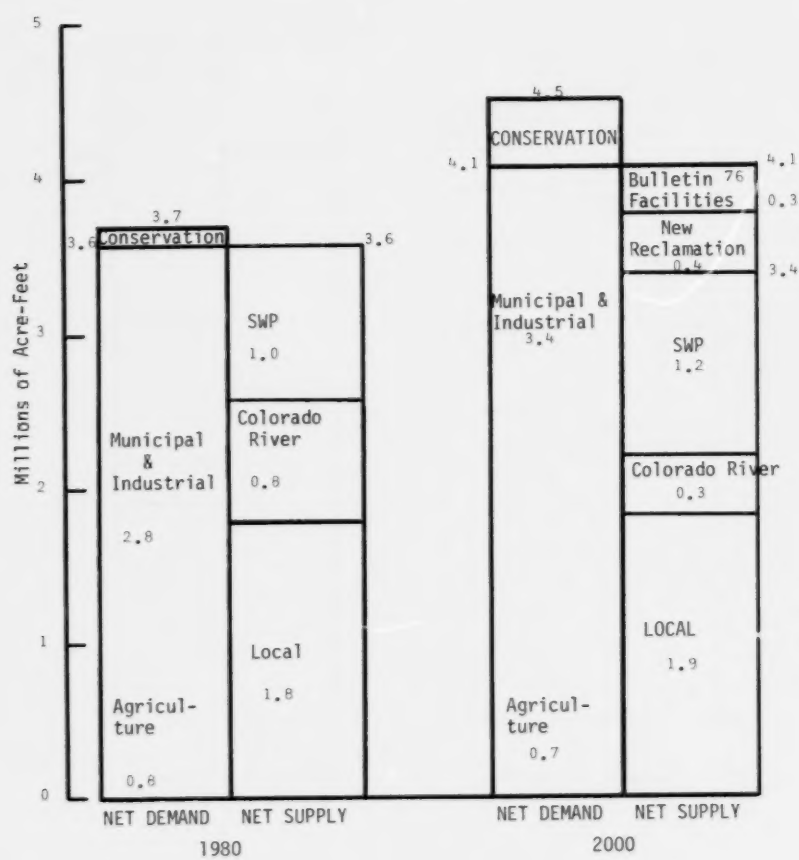


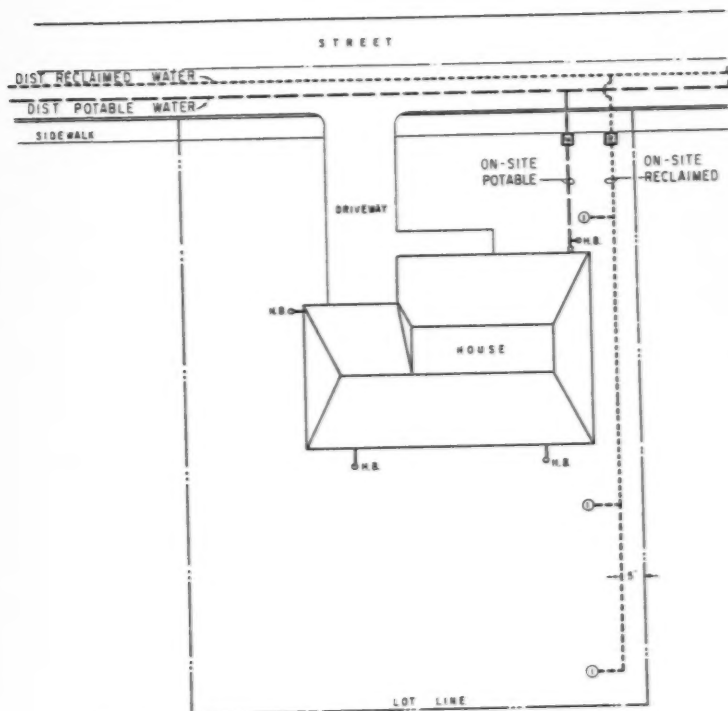
FIGURE 2

WATER QUALITY COMPARISONS

Constituent	Units	Drinking Water			Treated Effluent Rqmts		Tapia Effluent
		Recom. Max. Contaminated Level	Jensen Plant Treated Water	West-lake Res.	NPDES (CA0056014) Stream Standards		
					Ave.	Max.	
							Averages
COD	mg/l	----	----	----	----	----	35
BOD	mg/l	----	----	----	10	20	4
Suspended Solids	mg/l	----	----	----	5	10	4
Turbidity	TU	5	0.2	----	2	5	4
Tot. Coliform, 7 day median	MPN	----	----	----	2.2	23	<2
pH	----	----	8.0	8.0	----	6-9	7.4
Oil & Grease	mg/l	----	----	----	5	10	0.7
Settleable Solids	mg/l	----	----	----	0.1	0.2	<0.1
Tot. Nitrogen	mg/l	----	----	----	----	0.2	17
Tot. Dissolved Solids	mg/l	500	350	435	----	40	900
Chlorides	mg/l	250	55	58	----	1000	110
Chloride & Sulfate	mg/l	----	----	----	----	175	380
Arsenic	mg/l	0.05	----	----	----	500	0.01
Barium	mg/l	1.0	----	----	----	DWS	----
Boron	mg/l	----	0.16	----	----	----	0.8
Chromium	mg/l	0.05	----	----	----	1.5	0.005
Fluoride	mg/l	----	0.25	0.3	----	DWS	0.35
Copper	mg/l	1.0	----	0.2	----	1.2	0.02
Lead	mg/l	0.05	----	----	0.1	0.2	0.03
Cadmium	mg/l	0.010	----	----	0.02	0.03	0.004
Iron	mg/l	0.3	----	0.03	----	DWS	----
Mercury	mg/l	0.002	----	----	----	DWS	<0.001
Nickel	mg/l	----	----	----	----	DWS	0.03
Silver	mg/l	0.05	----	----	----	DWS	0.01
Cyanide	mg/l	----	----	----	----	DWS	<0.001
Zinc	mg/l	5.0	----	0.01	0.3	0.5	0.08
Sodium	mg/l	----	48	55	----	DWS	160
Manganese	mg/l	0.05	----	0.03	----	----	----
Phenols	mg/l	----	----	----	0.5	1.0	0.01
Selenium	mg/l	0.01	----	----	----	----	0.003
MBAS	mg/l	0.5	----	----	0.5	----	0.07
(Surfactants)	mg/l	----	----	----	0.1	----	<0.1
Residual Chlorine	mg/l	----	----	----	----	----	0.001
Tot. Chlorinated Hydrocarbons	mg/l	----	----	----	----	----	150
Calcium	mg/l	----	46	39	----	----	30
Magnesium	mg/l	----	17	19	----	----	15
Nitrate as N	mg/l	10.2	0.3	0.5	----	----	0.8
Nitrite as N	mg/l	----	----	0.02	----	----	270
Sulfate	mg/l	250	100	85	----	----	29
Phosphate as PO ₄	mg/l	----	----	0.4	----	----	0.0
Polychlorinated Biphenyls	mg/l	----	----	----	90	----	99
Toxicity	%Surv.	----	----	----	----	----	5
Radioactivity	PC/l	15	----	----	----	----	15
- alpha	PC/l	50	----	----	----	----	1100
- beta	PC/l	----	----	----	----	----	----
Specific Conductance	µ mhos	900	600	340	----	----	----
Tot. Hardness as CaCO ₃	mg/l	----	185	160	----	----	----
Dissolved Oxygen	mg/l	----	----	----	Stream >5	----	7

Note: DWS = Drinking Water Standards

FIGURE 4

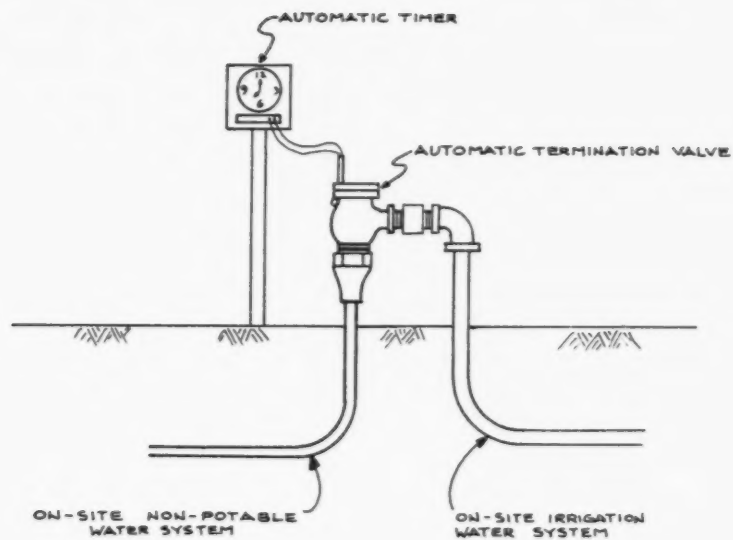


NOTES

- ① CONTROL VALVES TO IRRIGATION SYSTEM
- H.B. HOSE BIB (ALL HOSE BIBS FROM POTABLE WATER SYSTEM)
- WATER BOX - POTABLE POINT OF CONNECTION
- NON-POTABLE POINT OF CONNECTION

TYPICAL RESIDENTIAL LOT
DUAL WATER SYSTEM

FIGURE 5

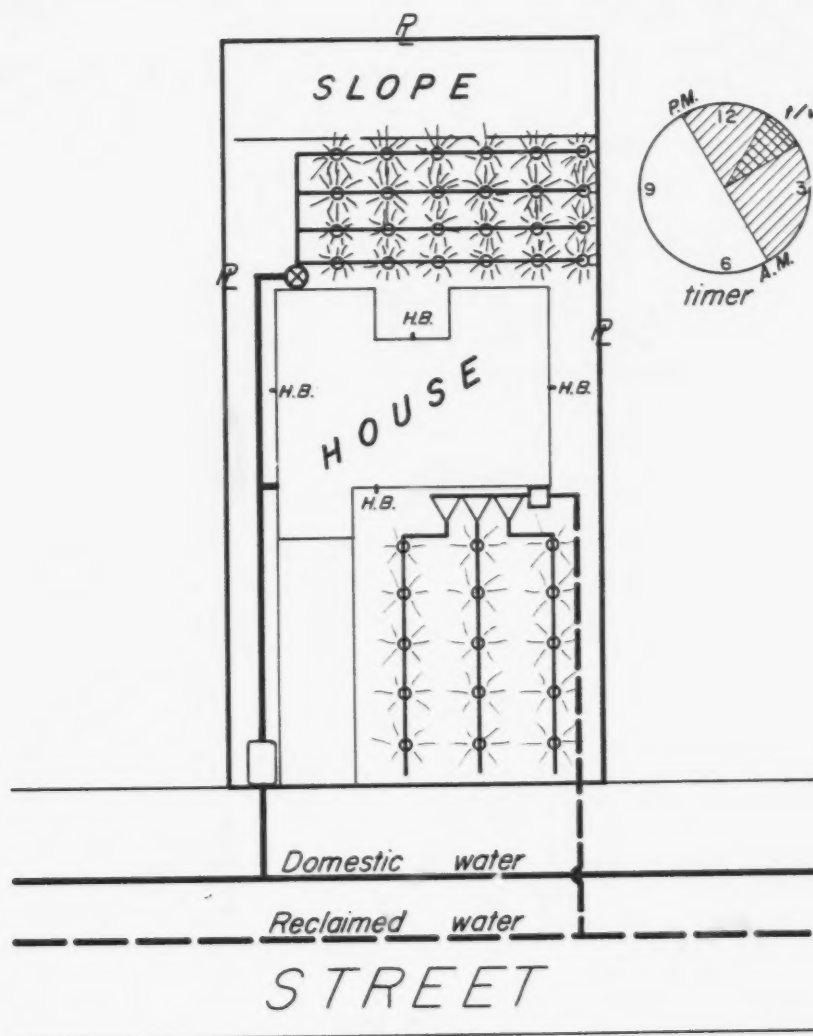


NOT TO SCALE

PLATE 1

**LAS VIRGENES
MUNICIPAL WATER DISTRICT
ON-SITE VALVE TERMINATION**

FIGURE 6



LEGEND

- meter
- ▽ timer/valve homeowner controlled
- timer, controlled by third party
- H.B. hose bib

FIGURE 7

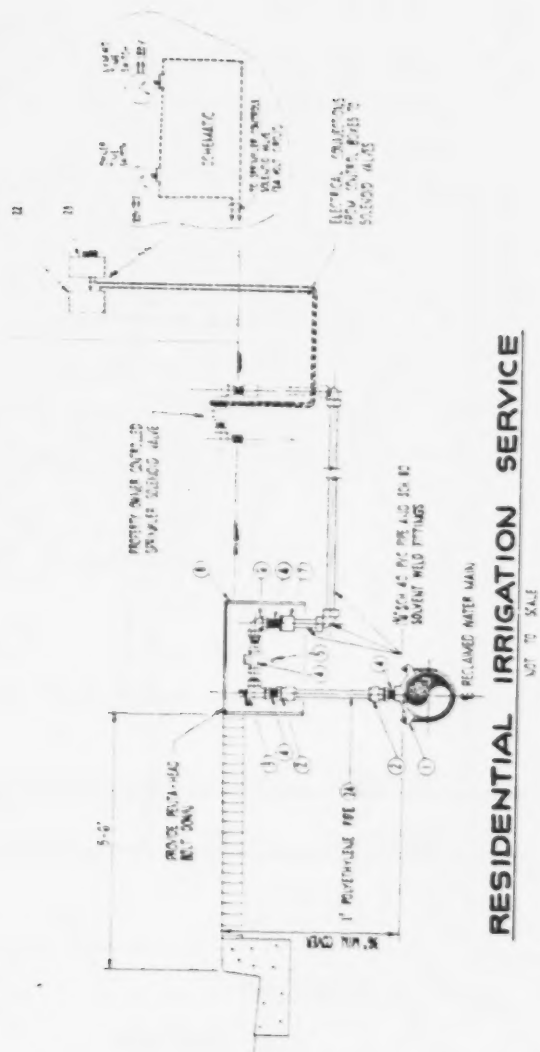


FIGURE 8

TYPICAL DUAL WATER SYSTEM IN SUBDIVISIONS

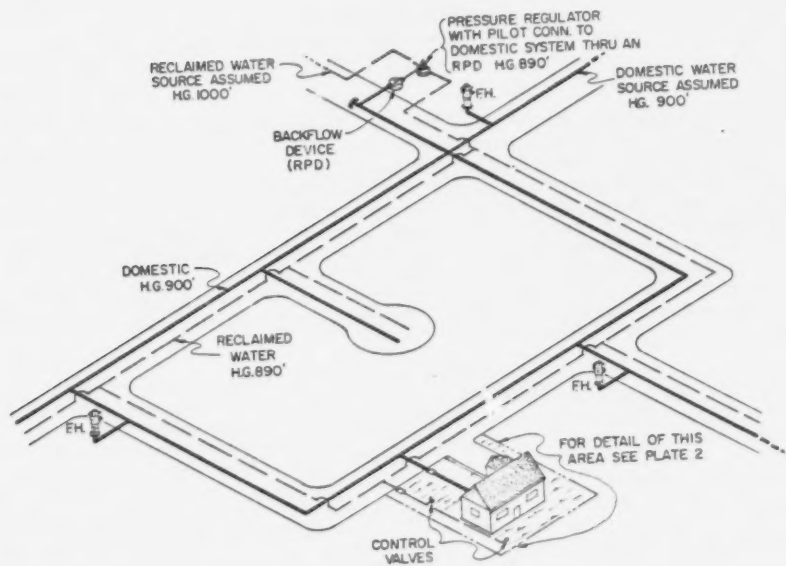


FIGURE 9

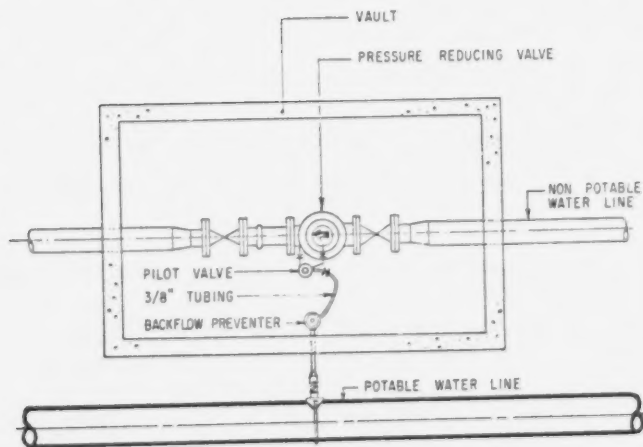


FIGURE 10
PRESSURE REDUCING STATION

IMPACT OF PRETREATMENT REGULATIONS
ON REUSE POTENTIAL OF MUNICIPAL WASTEWATERS

Jon C. Dyer, President
Environmental Technology Consultants, Inc.
Springfield, Virginia

Indirect industrial dischargers have utilized municipal wastewater treatment systems through the years and in most cases, with minimal amounts of pretreatment. These industrial wastes, both compatible and non-compatible, have been treated in conjunction with the municipal wastes. In many areas, severe operational problems and treatment facility upsets have occurred. Slugs of concentrated toxic pollutants have wiped out the biological populations required in biological treatment facilities. Pollutants may bypass treatment processes or pass-through the facility and be discharged directly with treatment plant effluent. Another problem, that of heavy metals, from some industrial processes can end up in waste sludges and effluents.

In many areas, the concept of reuse of municipal wastewaters has not been given serious consideration due to the quality of the wastewater and the unreliability of process control. Implementation of industrial pretreatment programs, prior to discharge into municipal systems, will improve the quality of municipal wastewater effluents, control process upsets, and assist in providing reliable treatment. Reliable treatment coupled with a positive local understanding will enhance the potential reuse of municipal wastewaters for irrigation, landscape development, and groundwater recharge.

NATIONAL REGULATIONS

National regulations, both existing and pending, pertaining to all facets of the environment are of concern to the successful development and implementation of wastewater reuse projects. The following major pieces of legislation impact both the industrial and municipal communities:

- The Federal Water Pollution Control Act, as amended, 1972 (PL 92-500)
- The Clean Water Act Amendments of 1977 (CWA) (PL 95-217)
 - General Pretreatment Regulation (40 CFR, 403)
 - Section 405 - Sludge Regulations
- The Resource Conservation and Recovery Act of 1976 (RCRA) (PL 94-580)

In addition, an important court decision, the National Resources Defense Council (NRDC) Consent Decree has also influenced EPA's pollution control efforts.

These pieces of legislation will enable EPA to "close the circle" of environmental control within the United States. A brief discussion of each follows:

The Federal Water Pollution Control Act (PL 92-500)

The 1972 Amendments (PL-92-500) was a complex law covering a wide range water pollution control regulations. Both the municipal and industrial communities were impacted significantly. Mandatory secondary treatment for publicly owned treatment works (POTW's) and a base level of pollution control, Best Practicable Control Technology Currently Available (BPCTCA) installed by direct industrial dischargers to waterways by July 1, 1977 highlighted the requirements. PL 92-500 also required EPA to establish levels of pollution reduction achievable by industry using Best Available Technology Economically Achievable (BATEA/BAT) by 1983. In addition, a new source standard calling for the use of advanced technology was required.

EPA standards are to be met nationally, regardless of location. Where State requirements or water quality requirements dictate, more stringent standards can apply. To enforce these requirements, PL 92-500 set the framework for the National Pollutant Discharge Elimination System (NPDES). NPDES permits are required for each point source discharge into waterways; this includes industrial and municipal sources.

PL 92-500 contained a special section dealing with toxic substances. Effluent standards or total prohibition of these pollutants were to be established. EPA was also directed to prepare a list of designated toxic substances. (See Table 1.)

The NRDC Consent Decree. In 1976, after being sued by NRDC, EPA agreed to concentrate attention on potentially toxic substances using technology-based standards. Toxic substances were to be included in BAT Standards to be issued for 21 industrial categories (Table 2). The initial list of 65 substances has since been broken down into 129 specific substances. Substantial portions of this agreement were incorporated into the Clean Water Act.

The Clean Water Act Amendments of 1977 (CWA) (PL-95-217)

The Federal Water Pollution Control Act was amended December 28, 1977 and is known as the Clean Water Act (CWA) of 1977. These amendments involve numerous changes to PL 92-500 including control of industrial discharges. The basic thrust of CWA is to provide more emphasis for control of toxic pollutants.

General Pretreatment Regulation (40 CFR, 403). EPA promulgated on June 26, 1978 the general Pretreatment Regulation (40 CFR, 403). This regulation, mandated by the CWA, governs the control of industrial wastes introduced into Publicly Owned Treatment Works (POTW's). This pretreatment regulation establishes the framework for application and enforcement of technology-based Pretreatment Standards (published as separate regulations) and general Prohibitive Discharge Standards for indirect industrial or non-domestic dischargers. Objectives of the regulation are:

- Prevent introduction of pollutants into POTW's which will interfere with plant operations and/or disposal or use of municipal sludges.
- Prevent introduction of pollutants into POTW's which will pass through treatment works in unacceptable amounts to receiving waters or municipal sludge.

- Improve feasibility of recycling and reclamation of municipal and industrial wastewaters and sludges.

Section 405 - Sludge Regulations. EPA is developing regulations concerning the disposal of municipal wastewater treatment sludges. After much discussion, EPA has decided to eliminate municipal sludges from any direct RCRA requirements. The 405 regulations will contain requirements as strict or stricter for municipal sludges with similar characteristics to hazardous industrial wastes.

The Resource Conservation and Recovery Act of 1976 (RCRA) (PL 94-580)

The Resource Conservation and Recovery Act (RCRA) was enacted in 1976 and provides EPA with the ability to control and regulate all solid wastes, sludges, and hazardous residues from the "cradle to the grave". The two basic objectives of RCRA are:

- 1) Protection of public health and environment,
- 2) Conservation of natural resources.

Hazardous waste guidelines and regulations on identification and listing (sections 3001, 3002, and 3004) was proposed December 18, 1978 and is currently being reviewed. Under these proposed criteria, solid waste is considered hazardous if it is flammable, corrosive, reactive or toxic. These regulations are of importance to industry. EPA estimates there is a 80-90% chance that current methods utilized by an industry to treat, store, and dispose of its hazardous residues will be illegal by early 1980. EPA will be looking hard at on-site disposal operations by waste generators. EPA plans to issue 5-year interim permits for existing facilities.

Regulations covering hazardous-waste management (Subtitle C) and land disposal of non-hazardous wastes (Subtitle D) are of major interest. The following breakout lists major areas of concern within Subtitles C and D.

Subtitle C - Hazardous-Waste Management.

- Section 3001 - Identification and listing of hazardous wastes (Dec. '78) (ignitable, corrosive, reactive or toxic).
- Section 3002 - Standards applicable to hazardous-waste generators (Dec. '78) (records, labeling, containers, chemical composition, manifest system, periodically report quantities and disposition of wastes generated).
- Section 3003 - Standards applicable to hazardous-waste transporters. Proposed regs (Apr. 28, 1978) (records, transport labeled wastes only, manifest system, transport to permitted disposal site only).
- Section 3004 - Standards applicable to management of hazardous waste treatment, storage and disposal facilities (Dec. '78) (records; reporting, monitoring, inspection, compliance; history; location, design and construction of facilities; contingency plans, operation, maintenance, ownership, personnel and financial responsibility; permit compliance).

- Section 3005 - Permits for treatment, storage or disposal facilities. Effective 6 months after regulations are promulgated. (permit application, issuance, interim permits for facilities in existence as of Oct. 21, 1976 - complied with preliminary notification requirements and application filed prior to effective date of regulations).
- Section 3006 - Authorization of State programs. Federal guidelines to be published. In general, States will regulate.

Subtitle D - Non-hazardous Waste Management - State or Regional Solid Waste Plans.

Section 4004 - Guidelines for plans. (Environmentally sensitive areas such as: wetlands, etc.; surface water; groundwater; air; land application consideration such as pathogens, Cd, etc.; disease vectors; safety).

Summary

States play an important role in the implementation of various Federal Regulations as discussed. Wastewater reuse projects can become reality instead of studies with the proper positive State involvement. In regard to pretreatment, NPDES States are required to develop authorities, procedures and resources to coordinate Pretreatment Programs applicable to the State's particular needs. Three options are available to an NPDES State in meeting the requirements of implementing a State Pretreatment Program. The available options follow. The option that appears to be reasonable and effective in most cases is option (1) where the State utilizes applicable POTW's to conduct local Pretreatment Programs and concurrently conducts a program for the balance of IU's not covered by POTW Pretreatment Programs. This option requires a coordination effort and maximizes use of existing personnel within municipalities with in-house knowledge of local conditions.

State Options

<u>State Program (1)</u>	<u>State Program (2)</u>	<u>State Program (3)</u>
Major POTW's have Programs.	Majority or POTW's have Programs.	Few, if any POTW's have Programs.
State assumes responsibility for a moderate number of IU's.	State assumes responsibility for a minimum number of IU's.	State assumes responsibility for a maximum number of IU's.
State coordination effort required - moderate resources.	State overview effort required - few resources.	Detailed State involvement required - maximum resource investment.

Generally, local Pretreatment Programs will be the primary vehicle for administering, applying and enforcing Federal Pretreatment Standards for Industrial Users (IU's) of POTW's. States will be required to apply and enforce Pretreatment Standards directly against indirect dischargers where local programs are not required or have not been developed.

EPA estimates greater than 3,000 POTW's will be impacted by these regulations. With the implementation and enforcement of these pollution abatement programs, municipal wastewaters will be more applicable to potential reuse. Priority pollutants will no longer be uncontrolled and through RCRA, illicit dumping of toxics into municipal systems will be eliminated. In the U.S., municipal wastewaters will truly be a valuable resource.

IMPACT OF PRETREATMENT

The City of Grand Rapids, (metropolitan population of 350,000), initiated a comprehensive industrial waste pretreatment program in 1969. The Grand River flows through the heart of the City and is an important recreational resource for western Michigan. The metropolitan area is highly industrialized. During the 1960's, industrial contamination of the Grand River created severe environmental problems (fish kills). Since one of the largest industrial sectors is electroplating, almost 40 companies provide services to the automotive-related industry.

The City Commission enacted a sewer use ordinance to control industrial discharges into the municipal system. Key pollutants of concern included cyanide and heavy metals. The City developed compliance procedures, a surveillance program and heavily involved the news media to develop public support for the program.

Significant reductions in cyanide and metal concentrations in both the wastewater influent and effluent have been noted since the pretreatment program and are depicted in Figures 1 and 2. The Grand River has been revived and its future as a recreational resource is almost unlimited.

Currently, Grand Rapids is expanding its activated sludge treatment capacity from 45 to 90 MGD. The portion of flow receiving full treatment is consistently meeting effluent characteristics of 10 mg/l of BOD and SS. This effluent is used internally within the plant grounds for irrigation, wash-down and clean-up purposes. Another point of interest in addition to the reduction in cyanide and metals in the plant influent is the reduction in industrial water usage. The Grand Rapids pretreatment program is responsible for approximately 40% reduction in water usage within the industrial community. This reduction has extended the life of the water treatment facility and eliminated the need for immediate expansion.

REFERENCES

1. BIENER, JAMES A. & BOUMA, WILLIAM H. An Industrial Waste Pretreatment Success Story, Public Works, (Nov., 1978).
2. U.S. EPA, General Pretreatment Regulations for Existing and New Sources of Pollution (40 CFR, 403), Federal Register, (June 26, 1978).
3. U.S. EPA, Office of Water Enforcement, NPDES Guidance on Implementation of Pretreatment Programs, (Feb. 12, 1979).

TABLE 1
65 Toxic Pollutants Listed in Consent Decree
and Referenced in 307(a) of The CWA of 1977

Acenaphthene	Endrin and metabolites
Acrolein	Ethylbenzene
Acrylonitrile	Fluoranthene
Aldrin/Dieldrin	Haloethers
Antimony and compounds	Halomethanes
Arsenic and compounds	Heptachlor and metabolites
Asbestos	Hexachlorobutadiene
Benzene	Hexachlorocyclopentadiene
Benzidine	Hexachlorocyclohexane
Beryllium and compounds	Isophorone
Cadmium and compounds	Lead and compounds
Carbon tetrachloride	Mercury and compounds
Chlordane	Naphthalene
Chlorinated benzenes	Nickel and compounds
Chlorinated ethanes	Nitrobenzene
Chloralkyl ethers	Nitrophenols
Chlorinated naphthalene	Nitrosamines
Chlorinated phenols	Pentachlorophenol
Chloroform	Phenol
2-chlorophenol	Phthalate esters
Chromium and compounds	Polychlorinated biphenyls
Copper and compounds	(PCBs)
Cyanides	Polynuclear aromatic hydro-
DOT and metabolites	carbons
Dichlorobenzenes	Selenium and compounds
Dichlorobenzidine	Silver and compounds
Dichloroethylenes	2,3,7,8,-Tetrachlorodibenzo-
2, 4-dichlorophenol	p-dioxin (TCDD)
Dichloropropane &	Tetrachloroethylene
Dichloropropene	Thallium and compounds
2, 4-dimethylphenol	Toluene
Dinitrotoluene	Toxaphene
Diphenylhydrazine	Trichloroethylene
Endosulfan and metabolites	Vinyl chloride
	Zinc and compounds

TABLE 2
Number of Indirect Dischargers
for The 21 Consent Industries

Industry	Indirect Discharges	
	Total	Total Number To Be Regulated By Pretreatment Reqs.
Auto and Other Laundries	90,000	1,500
Coal Mining	0	0
Electroplating	16,086	16,086
Inorganic Chemicals	120	108
Iron and Steel	238	237
Leather	170	170
Machinery and Mechanical Products		
Mechanical Products	71,000	3,000
Battery Manufacturing	170	170
Plastics Processing	2,700	1,350
Foundries	880	880
Coil Coating	50	50
Porcelain Enameling	138	138
Aluminum Forming	70	70
Copper Products	57	57
Electric & Electronic	8,000	500
Photographic Supplies	110	110
Miscellaneous Chemical Manufacturing		
Pesticide Manufacturing	46	37
Gum and Wood Chemicals	7	6
Pharmaceutical	200	160
Explosives	0	0
Adhesives and Sealants	500	500
Carbon Black	1	1
Nonferrous Metals	79	57
Ore Mining	0	0
Organic Chemicals	900	720
Paint and Ink	1,600	460
Printing and Publishing	63,000	2,300
Paving and Roofing	138	0
Petroleum Manufacturing	48	48
Plastic and Synthetic	160	128
Pulp & Paper	260	50
Rubber	537	9
Soap and Detergent	450	400
Steam Electric	150	150
Textile Mills	1,700	915
Timber Products	189	49
TOTAL	259,754	30,416

NOTE: Numbers shown are best available estimates, 1/15/79, based on best available information and are probably maximum.

FIGURE 1
Total Cyanide in Wastewater
Grand Rapids, Michigan

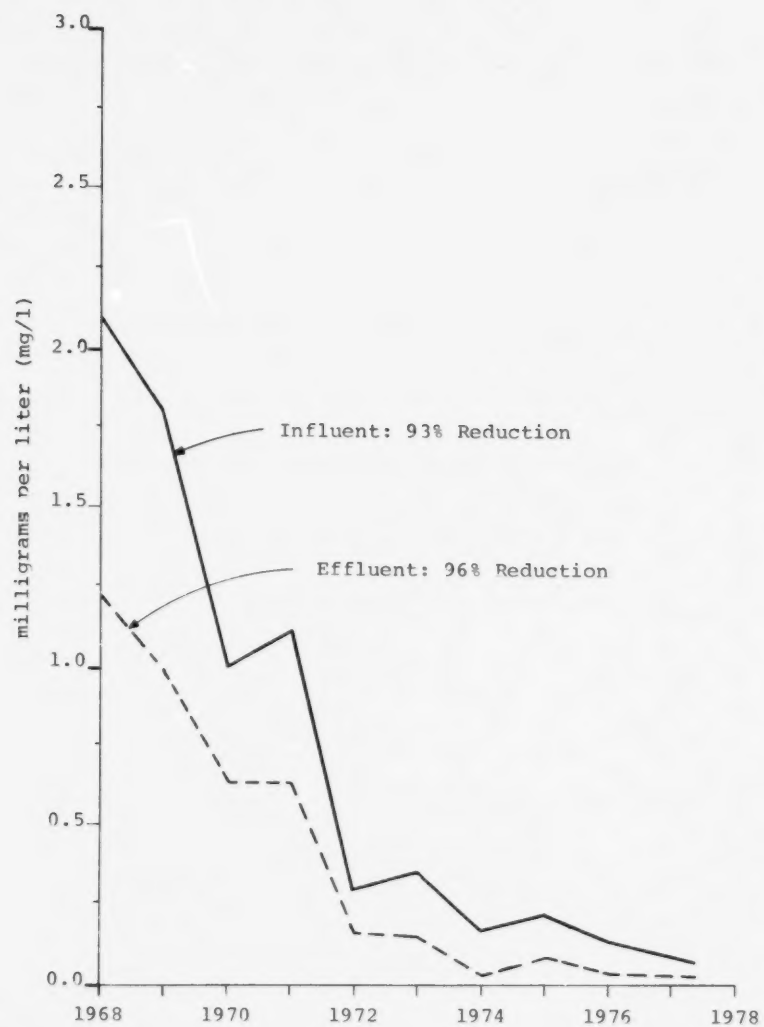
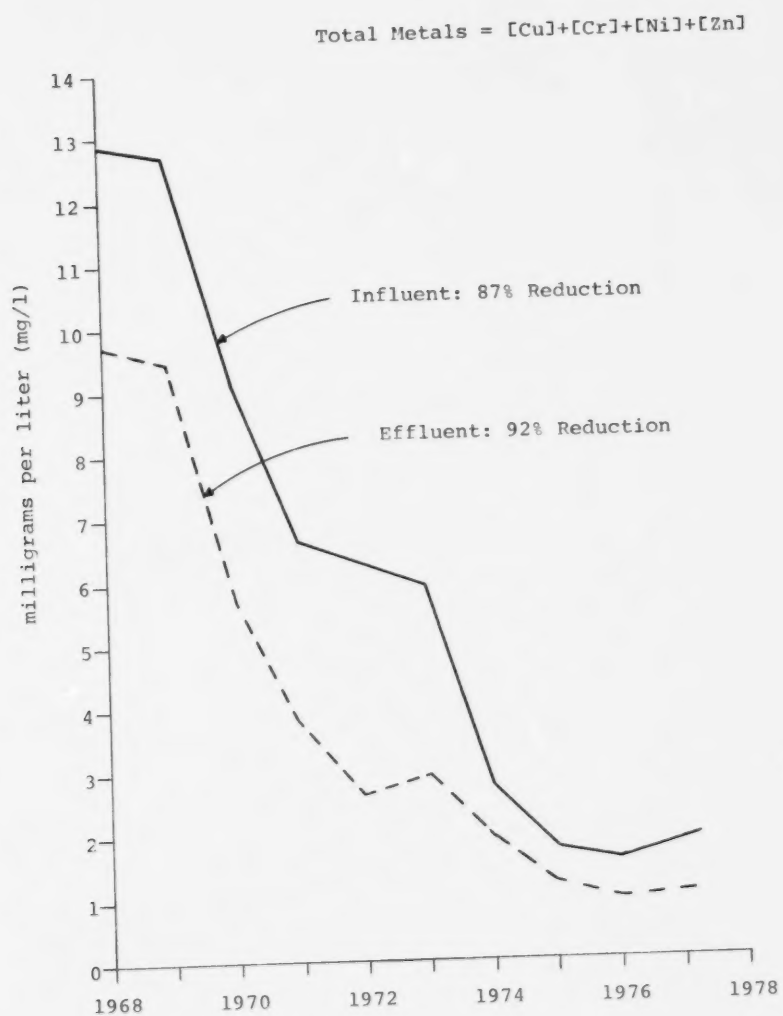


FIGURE 2
Total Metals in Wastewater
Grand Rapids, Michigan



IRVINE RANCH WATER DISTRICT
TURNS RESEARCH TO APPLICATION

William F. Hurst
Irvine Ranch Water District

Garry E. Lee
Lowry and Associates

Dr. Jan Scherfig
University of California, Irvine

Perhaps no water purveyor better epitomizes this symposium's theme-- "Water Reuse, From Research to Application" -- than the Irvine Ranch Water District (IRWD) in Southern California. IRWD's "Total Water Resource Management" program includes a dual irrigation system which uses reclaimed water as the primary source while untreated imported water, groundwater and the area's small amount of surface runoff fulfill only supplemental roles in many parts of the growing District.

What makes this all the more significant among today's nation-wide efforts to implement reclaimed water programs is that IRWD is a coastal district which has resisted the ocean disposal approach of neighboring agencies and has chosen to reuse virtually all of its wastewater.

The major emphasis of this paper is that reclaimed water can be a viable supplemental source of water and should be considered very seriously by water purveyors. The lessons learned here can be extremely valuable in implementing similar systems elsewhere.

IRWD'S UNIQUE CIRCUMSTANCES

A unique set of circumstances accounts for much of IRWD's success in implementing its complete wastewater reuse program. A major benefit was that a single landowner--The Irvine Company--owned most of the property in the district when IRWD was formed in 1961. Thanks to the forward-thinking developer, district staff and consultants, a futuristic water system was envisioned for the district's sprawling ranch lands. This provided ample lead time for planning the new communities.

Large irrigation demands of the Irvine Ranch also meant that IRWD effluent could be put to beneficial use rather than simply disposed of. As the cost of imported water continued to rise, reclaimed water became increasingly attractive to the Ranch. At the same time revenues from the sale of reclaimed water enabled IRWD to offset the cost of new irrigation facilities. Other advantages were the absence of existing water facilities to abandon or modify; timing that coincided with emerging federal and state pollution control standards and water conservation trends; and research in wastewater reclamation that was at an advantageous stage.

Based on these circumstances, a series of studies evaluating the potential for reclamation has each time concluded that reclamation, including a dual water system, is cost competitive with disposal and a single, potable system. Still, feasibility--especially in a new area--is of little use without a key ingredient: commitment. To this

end, IRWD's Board and staff strongly supported the dual system concept and, after extensively studying its feasibility, made the commitment to implement total reclamation in 1972.

TIMING AND PATIENCE

IRWD's planning and development lagged slightly behind demonstration projects in other areas of Southern California. This made implementation at IRWD much easier, since the technical and economic feasibility of treatment processes necessary for reclamation had already been demonstrated. Even with the ideal conditions surrounding IRWD, it took a great deal of patience and perseverance to finally obtain the necessary approvals. In fact, there are still obstacles--primarily the form of new regulatory requirements--that may continue to be an issue for some time.

BACKGROUND

When the District was created in 1961, the area was mostly agricultural with a population of only about 300. Impending development, which was at that time in the early planning stages, afforded IRWD one of its unique advantages--time for planning a comprehensive, long-range program. By 1965, when the District expanded its services to include wastewater treatment, IRWD had departed from traditional separate system concepts and began developing its total water management approach. IRWD had constructed its own treatment plant by 1968 and was delivering all of its effluent to the Agricultural Division of The Irvine Company for irrigation purposes.

During this period, IRWD was developing its first Irrigation Water System Master Plan. The most significant milestone occurred in 1972 when the Water Resource Master Plan was prepared. This plan considered the wastewater cycle (collection, treatment and "disposal" via reuse) along with potable water supply as integrated parts of the total water system.

In 1972, IRWD was producing 3 mgd of secondary effluent which was used for irrigation of farmlands and one golf course, and the time had come for IRWD to make a major decision about the future of its reclamation program. Two basic concepts for reuse and/or disposal of IRWD's effluent had developed over the years and were presented in the 1972 master plan. Alternative 1 called for primary dependence on reclamation and reuse, with ocean disposal limited to non-reclaimable industrial and other wastes; while Alternative 2 was primary dependence on ocean disposal, with reclamation and reuse just playing a supporting role.

Selection of the best alternative was based on two related points. First, water quality requirements for ocean disposal had become nearly as stringent as for reuse. Second, total reclamation appeared to have a lower overall cost, resulting from reclaimed water sales and reduced imported water costs. The District adopted a total reclamation objective in 1972 and soon thereafter authorized expansion of its wastewater reclamation facilities to 15 mgd with the understanding that, as the many institutional rough spots were ironed out, the reclamation program would be expanded. Because it coincided with the 1970's with emerging national water pollution control objectives, IRWD's commitment to total reclamation has gained increasing national attention.

REUSE REQUIREMENTS

In California, water quality and reuse requirements are regulated by the State Water Resources Control Board (SWRCB) along with its nine Regional Water Quality Control Boards (RWQCB) and the California Department of Health. A reuse permit is issued by the Regional Boards when requirements of Title 22 of the California Administrative Code, SWRCB, RWQCB and Health Department policies and guidelines have been met. In October 1977, IRWD obtained a "producer/primary user" permit from the local Regional Water Quality Control Board. The first such permit ever issued on as large a scale in the state, this permit allows IRWD to distribute reclaimed water to parks, schools, playgrounds, restricted recreational impoundments and for all agricultural uses except crops that are spray irrigated and eaten raw. The most restrictive requirement in the permit is for surface irrigation of food crops and the irrigation of parks, playgrounds and schoolyards. Essentially, the permit requires that "...the median number of coliform organisms in the effluent does not exceed 2.2 per 100 milliliters, as determined from the bacteriological results of the last seven days for which analyses have been completed." These standards are met by the IRWD plant.

From a farmer's standpoint, water quality requirements depend on the types of crops and their tolerance to dissolved minerals. All but a few of the many varieties of crops grown in the Irvine area do well on IRWD effluent. The relatively high total dissolved solids (TDS) content of about 900 parts per million (ppm) currently in IRWD's effluent is balanced by the use of a small amount of additional water for leaching. As larger amounts of lower TDS Northern California water are supplied, the leaching requirements will be less.

The impact of the percolation of relatively high TDS water into the groundwater basin has been considered by the Regional Water Quality Control Board, which has set a 720 ppm limitation. The Regional Water Quality Control Board also has an incremental limitation not to exceed 200 ppm between potable supplies and effluent. The groundwater impacts will soon be evaluated in greater detail, but could become an obstacle to IRWD's total water reclamation concept in the future.

THE TREATMENT PROCESS

IRWD currently treats and reuses 5.5 mgd of wastewater in its tertiary plant. An expansion project under construction will increase the plant capacity to 15 mgd by late 1979 or early 1980, although flows will not reach this level for some time.

Sewage reaching IRWD's plant is first shredded and degrittied to remove non-biodegradable substances and heavy particles. The degrittied sewage then flows to primary clarifiers where, during a regulated slow flow, 40 to 60 percent of the remaining suspended solids settle out. In this state, air is drawn by blowers from beneath the clarifier covers to dissipate odors.

Primary effluent then enters the activated sludge aeration tanks. Effluent from the secondary clarifiers is then put through multimedia sand filters, chlorinated and finally released for pumping to two reclaimed water storage reservoirs. Some effluent is diverted en route for direct use, while the rest ends up in the scenic reservoirs which attract geese, duck and other wildlife.

As of June 1978, IRWD's dual system facilities included more than 50 miles of District constructed pipelines, in addition to pump stations and reservoirs. The District recently obtained bond authorization for \$995 million worth of future construction (1979 cost of \$300 million) to implement total water resource management. Of this amount, \$460 million has been set aside for the conventional potable water system, \$335 million for the sewerage system, and \$200 million for tertiary treatment and the irrigation system.

RECLAIMED WATER VALUE

For several years already, IRWD effluent has been used for golf course irrigation and to irrigate a county regional park within the District. During the 1976 through 1977 drought in California, the importance of IRWD's reclamation system came sharply into focus when a 16 mg man-made lake was filled while deliveries of potable water to a recreational lake in a nearby community were cut off and the lake remained two-thirds empty. Within the past four years, the first reclaimed water deliveries were made to residential parks and greenbelts, a city park and school grounds--the result of obtaining the reuse permit.

Service to the residential and public areas was especially gratifying because of the vocal desire for reclaimed water expressed by the community. A professional survey conducted in IRWD in 1976 determined that 92 percent of those polled supported the use of reclaimed water for irrigation of greenbelts and recreational areas. Another poll, this one state-wide, showed that less than three percent of the population would oppose the use of reclaimed water for irrigating parks and lawns; and that only 14 percent would be against the irrigation of vegetable crops, which is the highest planned use within IRWD. At a more grassroots level, IRWD has felt a mild degree of public pressure, in the form of requests from community associations and numerous articles in the press, to speed up delivery of reclaimed water to older communities not having a dual water system. Certainly, an important factor contributing to such support was a well-planned and executed public information and education program.

Timely service to newly developing areas is guaranteed by the installation of a dual pipeline system. Developers are required to install reclaimed water lines along with potable water lines. In addition to residential areas, major new industrial and regional commercial sites will be served. The dual system has also been expanded to serve the Irvine Campus of the University of California. The University is expected to consume 2161 AF per year ultimately, at an annual savings of over \$160,000 in water purchases.

As the population increases, larger amounts of reclaimed water will be available. Projections of future volumes of reclaimed water are difficult to determine because they depend on development rates which will continue to be influenced by economic, environmental and political conditions. As an illustration, since IRWD's formation, the maximum planned population of the City of Irvine has been scaled down from a high of 600,000 to present estimates of a maximum of about 350,000 due to changing attitudes of public decision makers. The element of uncertainty affects both the amount of reclaimed water to be produced (equal to sewage flow) and the demand for . . . Land use and flow projections are updated annually by IRWD.

Present estimates for maximum volumes of reclaimed water (sewage flow)

based on the best indications, are about 41,000 AF per year. The ultimate annual demand for all types of irrigation is expected to approach 44,000 AF. Of this, 28,000 AF is estimated for landscape irrigation and 16,000 AF for permanent agriculture. So the amount of effluent IRWD could ultimately supply for irrigation is about 93 percent of the eventual demand. The potential for industrial process water demands is still being analyzed, and will have an effect on the total reclaimed water balance.

REGIONAL SIGNIFICANCE

In Southern California, which must import over half its water, reclamation promises to yield significant dollar savings. Obviously, the philosophy of using water once and then disposing of it is changing. The Director of California's Department of Water Resources, Ronald Robie, echoed the theme of this conference when he said "a concerted effort must be made to move from the planning stage to the actual reuse of reclaimed water."

Further, recent California legislation has prohibited use of potable water by a public agency for greenbelt irrigation, golf courses and parks when suitable reclaimed water is available at a reasonable cost. Clearly, reclaimed water has been identified as a new water source which will help conserve potable water.

The coming years will require a decision by the Irvine Ranch Water District as to its ultimate wastewater management plan. Ocean disposal for some effluent is still an alternative for an effective "fail safe" system and is still being considered. At the present time, IRWD is developing a Management and Action Program which will identify key facilities and timetables for planning, design and construction over the next few years.

NATIONAL AND INTERNATIONAL SIGNIFICANCE

Generally, the experience gained by IRWD can be applied to other areas interested in reclamation. Market assessment is the first step in evaluating the reuse potential of any given area. In brief, the major areas of consideration are:

1. Identification and study of opportunities for reuse.
2. Identification of quality requirements for each type of reuse.
3. Groundwater considerations.
4. Soil suitability.
5. Basin objectives.
6. Alternative water supply
 - a. Quantity
 - b. Quality
 - c. Price
7. Mineral quality of effluent.
8. Public acceptance.
 - a. Public education
 - b. Public participation
9. User acceptance.
 - a. Price, demand and quality relationship
 - b. What portion of total budget spent for water
 - c. Relationship to user discharge requirements and internal reuse systems.
10. Reclaimed water value
 - a. Fertilizer
 - b. Replacement of potable use
 - c. Overall system reliability
 - d. Intangible or indirect benefits.

11. Dual system facility requirements
 - a. Treatment
 - b. Storage
 - c. Distribution
12. Financing
 - a. Revenue program
 - b. Pricing policy
13. Water rights
14. Legal assistance
 - a. Requirements for use where feasible
 - b. Economic incentives for users.

The assessment of the market potential should be strong. In other words, an apparent sizable volume should be identified along with clear economic feasibility.

Planning projects identified in the market assessment should begin as soon as possible since final implementation can take many years. As much as possible, evaluation should focus on priority projects which can be resolved quickly. Project planning must be based on sound technical requirements, cost estimates and implementation schedules.

COMMITMENTS

The implementing agency's governing board and staff have to firmly commit to implementation. This is not to say, however, that an unreasonable project must be supported. The board and staff must be willing to take a strong public position and lead the way. There must also be a commitment of planning funds. Finally, there has to be a commitment by the user.

All proposed projects must be closely coordinated with state and local agencies such as State Water Resources Agencies, state and local health departments, school and park districts. This support can be extremely valuable not only in obtaining the necessary permits but also in the public relations and education programs.

Assuming that a project is worth undertaking, it takes a conscious "public education" program. Get out and tell your story. A vital part of this education should begin during the first stages of the project by initiating a public participation program. Give the public an opportunity for input, and make sure their concerns are sincerely addressed.

To pay for capital facilities and generate adequate revenue, a sound financial plan must be developed. Familiarity with the Clean Water Grant Program as a potential source of funds is recommended.

SUMMARY AND CONCLUSIONS

The Irvine Ranch Water District has been successful in developing a supplemental water resource through its total resource management concepts. IRWD was fortunate in having some very unique circumstances which may not exist for every agency. However, even with these advantages, it has not been an easy task. It has taken a long time, a lot of hard work and persistence. The best advice for those pursuing reclamation is "Be committed to ultimate implementation and be patient for results."

RECLAIMED WATER DISTRIBUTION SYSTEM PLANNING--
WALNUT VALLEY, CALIFORNIA

Roger C. Bales
Project Engineer, Brown and Caldwell

Edmund M. Biederman
General Manager, Walnut Valley Water District

Gary Arant
Assistant General Manager, Walnut Valley Water District

Walnut Valley Water District, located in Los Angeles County, California, is currently planning to distribute reclaimed wastewater from the Pomona Water Reclamation Plant as an alternate source of supply in the greater Walnut Valley Area. This paper describes the year-long planning effort undertaken by the District to market this reclaimed water and to develop a feasible, implementation project that can be constructed with available federal, state and local funds.

Walnut Valley is primarily a suburban-residential area, with long-term sustained industrial growth occurring along transportation corridors in the center of the valley. As shown on Fig. 1, the 43 square mile (112 km²) study area is comprised of the communities of Walnut, Diamond Bar and Rowland Heights, and includes portions of the City of Industry, Hacienda Heights, La Puente, West Covina and Pomona. The area is now entirely dependent upon imported water from the Colorado River and from northern California. With a long-term statewide shortage in developed supplies and with anticipated increases in the cost of imported State Water Project water, it became prudent for the District to look toward alternate local sources of water. Because the study area has sufficient capacity in local potable water distribution facilities to meet demands through the year 2000, the primary motivation for potential customers agreeing to use reclaimed water was necessarily economic. Secondary was the realization that they would suffer cutbacks in the event of another statewide drought.

Presently, there are approximately 235 (379 km) miles of water mains that distribute water from the source of supply to customer meters. The proposed separate reclaimed water distribution system will involve about 20 miles (32 km) of pipeline and will serve both medium-size users such as industries and golf courses and small landscape irrigation users at schools and parks. Four water agencies are involved in project implementation, whereas over eight local water agencies were involved in project planning.

Background of and Motivation for Project

Local pioneering projects, originating as early as 1928, helped to create an awareness of reclaimed wastewater as an

In 1969, the Walnut Valley Water District's Board of Directors authorized a preliminary feasibility study for developing a local reclaimed water system. The investigation identified potential reclaimed water users and demonstrated that there was a viable market for the tertiary treated wastewater available from the Pomona WRP. Following the study, the District began pursuing state and federal funding for construction of a reclaimed water system. In 1977, this latest engineering study and comprehensive market assessment was funded under the California Clean Water Grant Program.

The District's motivation for pursuing the project at this time is based on two factors. First is the provision of a low-cost supply and second is the continuous, long-term provision of sufficient water of adequate quality. Economically, this project will benefit not only the public and private entities directly using the reclaimed water by providing a low-cost supply but the overall community as well. As reclaimed water use is expanded, demand on domestic supplies will be reduced, thus helping to defer or offset further capital investment in the domestic water system, which is supported by local taxpayers and water system users. On the second issue, the statewide water shortage from 1972 through 1974 was about 2.5 million acre-ft ($3.1 \times 10^9 \text{ m}^3$) per year. Water conservation, partially offset by increased demands, could halve this shortage by the year 2000, but the remainder must be met by increased supplies. During the 1975-77 California drought, local agencies and large water users were keenly aware that although they had more than adequate legal entitlements and pipelines to deliver water imported from the Colorado River and from the State Water Project, they would have to cut back use even more if the drought continued or if a drought period came in the future when demand would be greater and Colorado River water less available. During the recent drought, the state used surplus Colorado River water that will be committed to the Central Arizona Project in the future. This project is thus based on the District's recognition that coordinated local efforts are necessary to develop reclaimed water to both meet statewide water shortages and assure the availability of sufficient water within its boundaries.

Preliminary Market Assessment

It was quite timely to commence marketing a readily available, alternate, low-cost water supply just as the state was ending its third year of drought and many northern reservoirs were nearly empty. The marketing approach for the reclaimed water was, however, that of selling a new product -- a product that although tested, still had many unanswered questions and was not well accepted by the general public as being a safe, desirable commodity. The history of reclaimed water use in the area by the Northside Water Company, which had used wastewater effluent for citrus irrigation since 1928 and continued to deliver effluent for alfalfa and park irrigation, contributed to public awareness of the potential applications of the supply. Inquiries had been made at the Walnut Valley Water District regarding

extension of the reclaimed water service to the local school districts, other parks and golf courses. These specific customers had experienced hardships resulting from the recent voluntary water conservation programs that limited water use for landscape irrigation. Various homeowners also inquired into the availability of reclaimed water, but costs of complying with health department guidelines for on-site facilities make this use uneconomical. Initially, the reclaimed water marketing team was faced with three basic tasks:

1. Identify applications for reclaimed water in and around the study area, consistent with its availability, quality and reliability.
2. Inform potential customers or end users about reclaimed water.
3. Determine demand for, or willingness to purchase, reclaimed water.

This data formed the basis for an engineering/economic evaluation of reclaimed water distribution system alternatives, which led to the detailed market assessment and obtaining of firm commitments from potential customers for reclaimed water use.

The preliminary market assessment began with a review of the more than 15,000 customer records in the study area, and later, records in five other retail water agencies adjacent to the study area. Customers using an average of more than 50,000 cubic feet (1420 m³) of water per month (1.15 acre-ft per month) were generally considered to be potential users. Because some minimum investment is necessary to convert an existing water system to reclaimed water use, the potential savings afforded by purchase of reclaimed water rather than purchase of potable water must be high enough to recover capital over a reasonable period of time and/or realize an acceptable return on invested capital. The 1.15 acre-ft per month criterion was tested, based on the following assumptions:

1. The maximum achievable savings from purchase of reclaimed water in lieu of potable water would be \$75 per acre-ft (\$0.06/m³).
2. Demonstration of a three-year payback period, or alternately, a 15 percent or greater return over an indefinite term appeared to generally reflect the marginal cost of capital criterion used by industries to identify acceptable cost-avoidance projects.
3. The minimum customer conversion required to beneficially use reclaimed water will be \$2,000 to \$2,500 (100 feet (30 m) of pipe plus a backflow prevention device).

For purchase of 1.15 acre-ft (1420 m³) per month at \$75 per acre-ft (\$0.06/m³) less than the potable water price, the annual savings would be about \$1,033. With a 15 percent return on investment over three years, this annual savings would allow an initial investment of about \$2,300, which is within the estimated minimum in-house conversion cost.

To fully develop the potential for reclaimed water use by each of the potential users identified by the preliminary market assessment, a comprehensive field survey was carried out. In general, this was intended to develop the following information:

1. Specific planned use for reclaimed water.
2. Present and future quantity needs.
3. Timing of needs.
4. Quality requirements.
5. Reliability features needed.
6. Used water disposal problems.
7. Internal capital investment needed to accept reclaimed water.
8. Preliminary intent or desire to use reclaimed water.

Two methods were employed to develop this information. The primary method involved telephone calls and field visits to the over 50 potential users identified. Second, an ad hoc Wastewater Reuse Advisory Committee was formed, and adjacent water utilities and potential end users were invited to attend public meetings.

A key concern of many reclaimed water customers was plant and supply reliability. Although it was put forth that reclaimed water was an interruptible supply, subject to availability, the Health Department was asking customers to sever their potable water connection to their irrigation or other system when a reclaimed water connection was made. This was accepted by most end users, based primarily on the good reliability record for the wastewater reclamation plant. Scheduled plant down time is about one hour per month, and only one plant upset, 13 hours in duration, is recorded during the past six years. Tertiary effluent turbidity and suspended solids are consistently less than 2 units and 2 mg/l, respectively. To provide a level of service commensurate with the potable water, in terms of pressure, quantity and location of service, it was decided to offer service up to the customer's property line, and where possible, size lines to meet existing peak flows. In a few cases where the peak to average demand ratio was excessive, a tentative program to mitigate peaks was worked out with the customer.

Engineering Analysis

In parallel with the market assessment and user consultation tasks, it was necessary to conduct an engineering/economic screening to determine which potential end users could be served with water through a proposed pipeline network. This enabled the marketing team to focus efforts on those really viable markets, while eliminating from consideration those markets too small or too distant from the source of supply. Fig. 2 presents a cost-of-service versus demand, or potential sales, curve that illustrated the out-

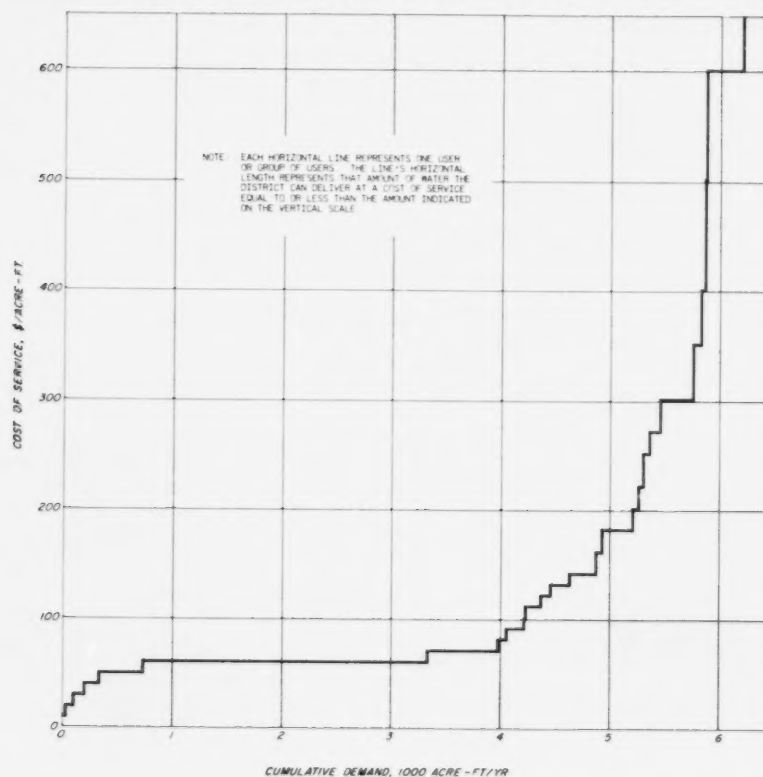


Fig. 2. Cost-of-Service vs Demand Curve

put of the computer screening tool used. As is apparent from the curve, there are a few key large users that form the economic basis for the system -- that is, their cost of service is much lower than the economic value of the service. Higher on the curve are smaller users, with a correspondingly higher cost of service.

Two basically different criteria are available to determine what size system is cost effective. The basis for comparison, called a baseline cost in the following discussion, refers to the net economic benefit, or true economic cost of the alternate supply. These criteria are:

1. Cost of service to each user on the system below baseline cost.
2. Average cost of service for system less than baseline cost.

The second criterion was used for all customers within the study area, as all can use water at the same elevation and it is consistent with present District water pricing policy. For customers outside the study area that will require incremental boosting, the first criterion was used.

Because this study was conducted under the Clean Water Grant Program in California, specific guidelines against which to test the project's cost effectiveness applied. The baseline against which cost effectiveness was judged was the long-run incremental cost of imported water. As there is a statewide shortage in developed supplies, the next most available, or easily developed, supply to meet present and future needs was used in the analysis. Were there no water shortage, it would be appropriate to use the cost of presently-developed marginal supplies. This approach is consistent with both the water quality factors outlined above and with the state's plan for long-term water development. Five cost elements associated with delivery of incremental supplies of treated water to the study area and distribution through the existing potable water network to reclaimed water markets are as follows: Costs are expressed in dollars per acre-ft ($\$/m^3$).

1. Development and delivery to Delta	\$158 (0.128)
2. Construction and operation of peripheral canal	4 (0.003)
3. Delivery to Southern California	154 (0.125)
4. Treatment	13 (0.011)
5. Local transmission and distribution	<u>121 (0.098)</u>
Total	\$450 (0.365)

Items 1 and 2 allow for construction and operation of new water conservation facilities to make additional water available to the existing California Aqueduct. Item 3 is the operating cost for the aqueduct and associated pumping plants. Numbers shown are long-run incremental costs (equivalent annual costs over a 20-year period for delivering incremental supplies).

Detailed Market Assessment

It was recognized that construction of a second pipeline network in the area for distribution of reclaimed water was essentially a speculative venture, as the District had no firm legal recourse to force use of reclaimed water in lieu of potable water. Recent state policies have strengthened water agencies' ability to encourage wastewater reuse, but actual use of reclaimed wastewater has been on a voluntary, economically motivated basis to date. Thus efforts during the detailed market assessment focused on seeking firm, voluntary commitments from potential end users on which to justify construction of a distribution network. Conditions the District addressed in the commitment letters were essentially the same as those outlined in the state's Policy and Action Plan for Water Reclamation in California. A typical commitment contained the following:

1. Minimum effluent quality guarantee, based on the Regional Water Quality Control Board effluent discharge requirements.
2. Caution that supply is interruptable and is subject to availability.
3. Maximum price, relative to the potable water price, that will be charged.
4. Specific application for water.
5. Annual quantity and variation of use.
6. Term of use.
7. Point of use regulations and restrictions, as promulgated by the Los Angeles County Department of Health Services and by the District.

Of paramount concern to end users was item 3, the maximum price that will be charged. The marketing team's approach was to determine the minimum discount, or savings, that end users would be willing to accept by assessing the on-site costs they were faced with to take the service from the property line to their irrigation or process water system. The marketing team worked closely with the County Health Department to define backflow prevention requirements at each site in the study area contemplating use of reclaimed water, to assure that all on-site customer costs were identified up-front before any commitment was made. In general, the following were required:

1. Approved reduced-pressure type backflow prevention device on potable water connection, with no cross connection between the potable and reclaimed lines.
2. Accurate, up-to-date drawings of the water system that show points of water use.

3. Responsible water system supervisor with proper training and knowledge of backflow prevention system operation.
4. Special hose bibbs or removal of hose bibbs.

All requirements were confirmed, in writing, prior to use commitments being made.

For each potential user contacted as part of the market assessment, a savings-demand relationship was estimated -- that is water use related to the savings in cost between potable water and lower cost reclaimed water.

This potential savings represents the maximum capital investment a user can make to convert his system to use reclaimed water. Expressed in mathematical form:

$$S = \frac{C}{D \times N}$$

where S is the required minimum savings in dollars per acre-ft, C is the user's internal capital investment, D is the user's demand for reclaimed water in acre-ft per year and N is the number of years within which the investment is to be recovered. This approach implicitly considers return on investment assuming that savings will continue beyond the pay-back period; most potential users had no set rate of return criteria for this type of investment.

The curve generated from this approach was a savings-demand curve, illustrated on Fig. 3. Each horizontal line represents a single user, plotted at his required savings level; the distance of each line along the demand axis represents the cumulative demand for water at a savings less than or equal to the stated value. This concept is also useful in considering that not all the users shown are paying the same price for water, due to the difference between potable water purveyors in their rate structures and levels.

Even with all technical factors showing a positive potential for use, the actual intent or desire to use reclaimed water was what determined whether the user was willing to put forth a long-term commitment. For various reasons, certain potential users declined to commit to reclaimed water use. Some of the factors that prevented this commitment being made were:

1. Probable inability to prevent intimate worker contact with reclaimed water.
2. Concern over water quality requirements.
3. Possibility of employee union problems.
4. Inability or reluctance to make any capital investment.

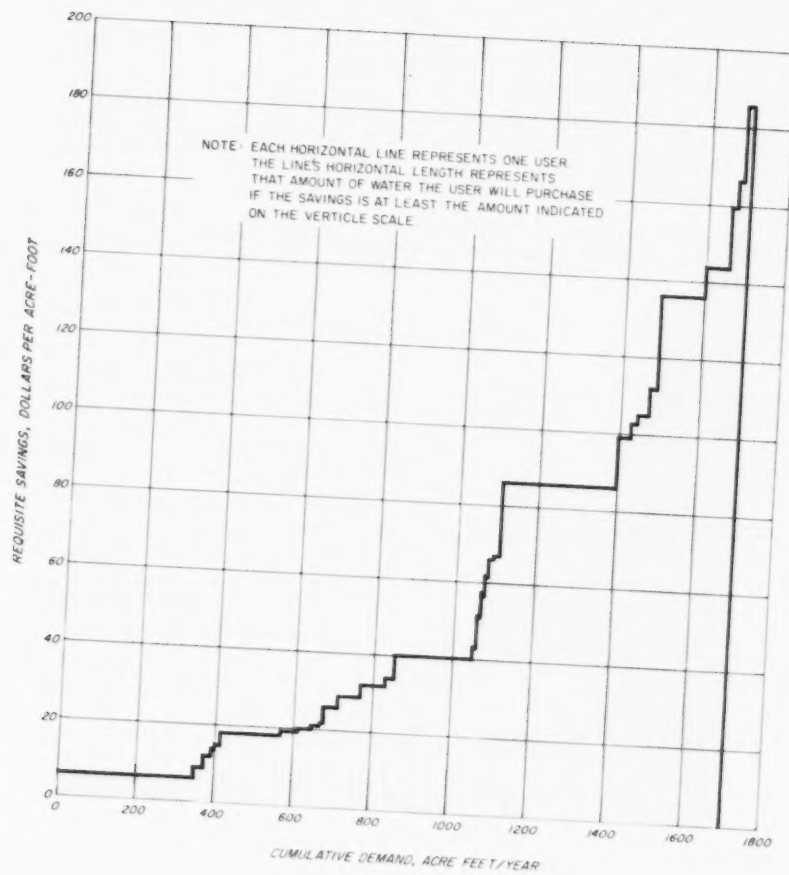


Fig. 3. Savings-Demand Curve

5. General feeling that reclaimed water demand is too low to justify making conversion.

The detailed marketing effort involved over 50 potential customers, many with multiple points of use, in eight retail water agencies. Four cities, as well as large areas of unincorporated land, were located in the study area.

Direct contact was made with potential reclaimed water users in other utilities' service areas only with the close cooperation of the adjacent utility, as state law prohibits a water purveyor from providing similar service or duplicating facilities to serve water to customers in another water utility's service area without the consent of the other utility. Cooperation of adjacent utilities in identifying, evaluating and eventually serving reclaimed water to potential users within their service area was thus essential. Excellent cooperation was freely given by the adjacent utilities.

There were no previous efforts of this nature or scope in the state on which to tailor the marketing approach, although similar efforts were being contemplated and are now underway. The approach taken by the marketing team was that each local water agency both has the responsibility and motivation to provide appropriate and sufficient service within its boundaries and has a strong identity and developed trust with its customers that must be exploited for the project to go forward from concept to reality.

Pricing Policy

The data developed during user consultations and presented on Fig. 3 formed the basis for the reclaimed water pricing policy. Based on these savings-demand relationships, it was decided to offer users a minimum \$40 per acre-ft ($\$0.032/\text{m}^3$) savings from the tail-block water rate as an incentive to sign letters of commitment for reclaimed water use. Demand decreases rapidly below a \$40 per acre-ft savings, whereas an increase in savings to over \$80 per acre-ft ($\$0.065/\text{m}^3$) is needed to attract additional large users. Further, a preliminary financial analysis indicated that the District would be in a position to offer a \$40 per acre-ft ($\$0.032/\text{m}^3$) savings.

By knowing one additional item about each user--what the user is now paying for water that would be displaced by reclaimed water--the savings-demand curve can be translated into a price-demand curve. Because most reclaimed water users will still require potable water, the reclaimed supply must be considered to replace the lowest price water or water purchased on the tail-block rate. In 1977, this ranged from \$0.30 to \$0.38 per hundred cubic feet or \$131 to \$160 per acre-ft ($\$0.106$ - $0.130/\text{m}^3$). In the first year of project operation, 1981, the tail-block rate is projected to be over \$200 per acre-ft ($\$0.162/\text{m}^3$).

Following development of a recommended project, a final reclaimed water pricing policy was outlined:

1. Price to end user at least \$40 per acre-ft (\$0.032/m³) less than potable water price.
2. Price is sufficient to recover all costs of the reclaimed water system, including source of supply, capital, operation, and maintenance and administration.
3. Price includes a margin or profit that may be accumulated to finance system expansion.

As illustrated on Fig. 4 for one service area, these components are present in the District's recommended pricing

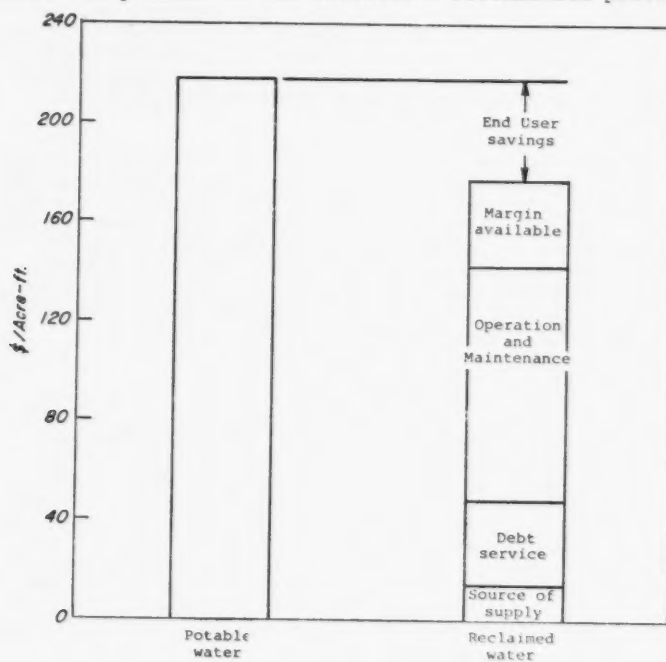
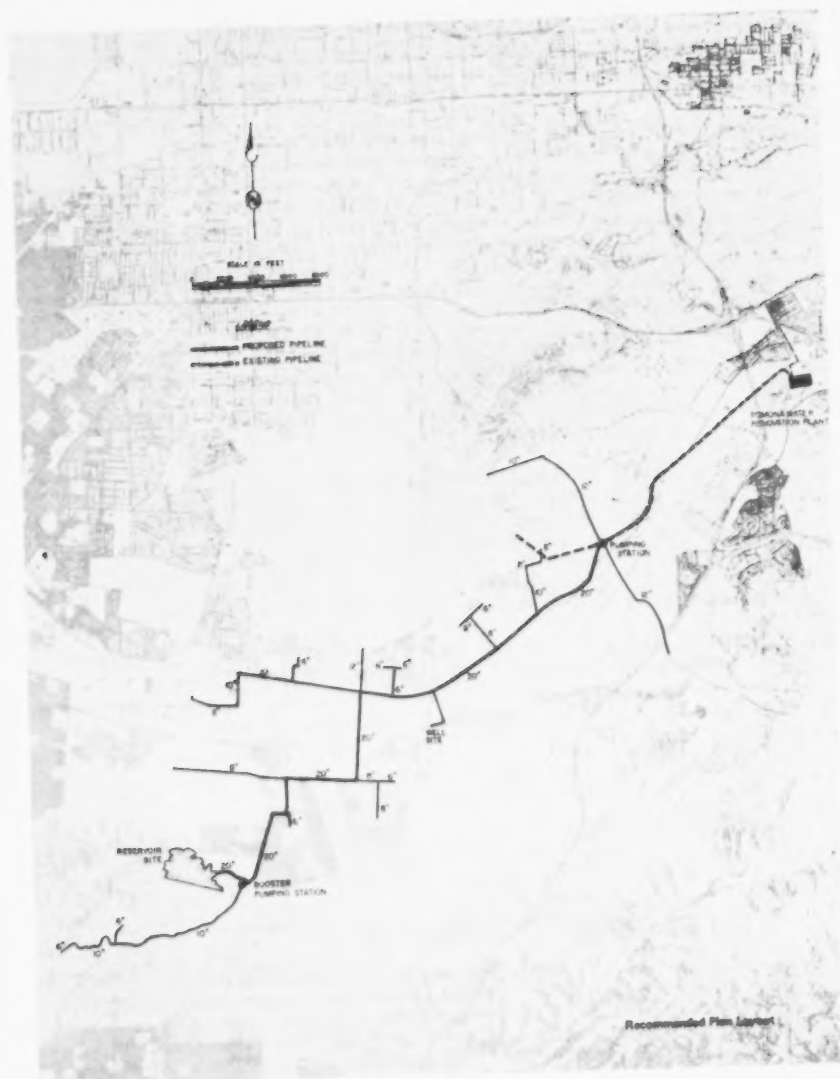


Fig. 4. Pricing Policy

policy. An alternate pricing policy could include a subsidy, which, for example, could come from the potable water system revenues.

Recommended Project

The recommended Walnut Valley Reclaimed Water distribution system, shown on the following map, includes some 20 miles (32 km) of pipeline, one reservoir, two pumping stations and two groundwater wells. Some 21 points of use



will be connected to the system initially with the flexibility to add future connections as the need arises. Total system construction cost, at current price levels is estimated to be about \$5.6 million. This represents an equivalent annual unit cost of about \$266 per acre-ft (\$0.215/m³), including operation and maintenance. Compared to the long-run incremental cost of \$450 per acre-ft (\$0.365/m³), reclaimed water development appears very favorable.

A 2,000 acre-ft (2.5 x 10⁶ m³) per year project is small compared to the long-term statewide water shortage of 2.5 million acre-ft (3.1 x 10⁹ m³) per year, or to California Governor Brown's goal of reusing 400,000 (0.5 x 10⁹ m³) acre-ft per year of wastewater by the year 1985, but it represents a logical step or building block in the regional and statewide water picture. Broad regional, state and federal support was responsible for construction of wastewater treatment facilities, and providing a source of supply will, in most cases, continue to depend upon this support for both construction and operation. Although this local project also depends on the same broader funding base, implementation and operation are only feasible at the local level by those agencies with a direct interest in providing water to their customers.

The concept of the local water purveyor as the sole agent that should retail water to customers was carried throughout facilities planning and forms a basis for implementation. Walnut Valley Water District, as lead agency, conducted the planning and is responsible for design and construction. Three other retail water purveyors will participate in the operation, but not the financing.

Fig. 5 illustrates the relationship of agencies that will be directly involved in project implementation. For

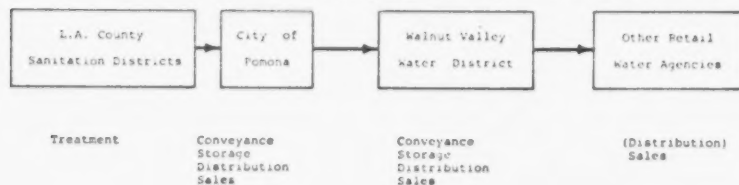


Fig. 5. Reclaimed Water System Structure

comparison, Fig. 6 shows the existing potable water structure. Whereas the Fig. 6 multi-level relationship is to facilitate distributing water from a large source over long distances to water retailers, the Fig. 5 relationship reflects a local supply passing through facilities owned and operated by water retailers. In this structure, the City of Pomona's Water Department acts as both a retail and wholesale agent for reclaimed water. Walnut Valley Water District will own and operate all facilities in the greater Walnut Valley area, but other agencies will be involved

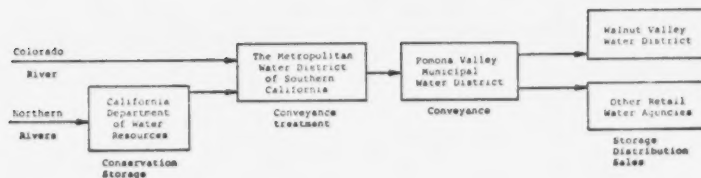


Fig. 6. Potable Water System Structure

with the District in actual water sales and customer service.

The primary concern among the retail water agencies was possible loss of revenue to pay fixed costs, should large customers join the reclaimed water system. This was particularly important where a flat commodity rate versus a declining block rate was in effect. Reclaimed water service will only be provided where revenues in excess of costs from the reclaimed water system are available to provide for the loss in potable water revenues, or where the growth in use of potable water offsets the loss in revenue. An additional concern applicable to two public utilities that are still contemplating offering reclaimed water service to their customers through this project was the possibility that the California Public Utilities Commission would decide that some facilities were no longer used or useful due to the provision of reclaimed water service, resulting in their exclusion from the rate base. Although the issue has not been formally tested with the Public Utilities Commission, it is recognized by the utilities that the potable lines are still needed, as the reclaimed water is an interruptible supply, subject to availability and does not possess a sufficiently high degree of reliability to be used for fire protection.

Financing Plans

The apparent availability of state and federal funds to partially offset construction costs for the proposed wastewater reuse system provides a tremendous boost to the financial feasibility of this local project. Because local water agencies are implementing the project as a parallel water distribution system, the financial analysis must demonstrate that water can be sold at a price less than that now charged for potable water. The economic analysis showed that total costs of reclaimed water are less than the total costs of developing an increment of imported water supply. However, the delivered cost of reclaimed water exceeds the average cost of potable water. This indicates that just as benefits from reclaimed wastewater extend beyond the local study area, project financing must also have a broader base in order for the local agencies to proceed. The presently-available source of funding that the District is pursuing is through the Clean Water Grant Program, based upon the project as a measure to mitigate

deteriorating water quality. A potential future source of state funds is through the State Water Project, based upon reclaimed water development as an element in the overall long-term state water plan.

An alternate for funding construction costs required in addition to grants could be the formation of a non-profit corporation, to issue lease mortgage bonds that would be secured by the project and repaid by revenues derived from the sale of the reclaimed water and augmented by additional funds taken from domestic water sales. Subsidizing the repayment of the bonds used to fund a portion of the reclaimed water system with domestic water revenues could be justified by several factors, one being that the construction of the reclaimed water system will partially alleviate the need for full capacity development of the domestic water system. Since many of the potential reclaimed water users place a large peaking demand on the domestic system, the substitution of reclaimed water for these peaking loads will allow the District to decrease the planned capacity of future transmission mains and reservoirs. The resulting reduced capital investment in the domestic system will be a benefit to all water consumers, both domestic and reclaimed.

A second consideration is that many of the potential reclaimed water users are public authorities (schools, cities and counties), who have large investments in ornamental landscaping, and the savings provided them by their using the lower-cost alternate source will be passed on in reduced costs to their supportive taxpayers.

These two benefits alone, which would be derived from the construction and operation of the reclaimed water system and passed on to the community, are sufficient justification to pledge a portion of domestic water revenues to finance this second source water distribution system.

Summary

This project proposes to develop an alternate source of low-cost supply, to the extent this alternate supply is available for use in the Walnut Valley area. It is being implemented at the local level by retail water purveyors who are responsible for the distribution of water to end users in their respective service areas. The project involves construction of a \$5.6 million distribution network of reclaimed water pipelines, pumping stations and a storage reservoir. Key to development of the recommended plan was a thorough and comprehensive engineering market assessment of potential water uses in the planning area. The end result of the market assessment was the obtaining of signed letters of commitment from end users to accept reclaimed water for specific long-term uses in lieu of potable water supplies. This unique element of the Walnut Valley facilities planning study distinguishes it from previous studies done in the area, which simply surveyed possible uses of the reclaimed water and conceptualized facilities needed

to serve those possible uses. The project is also unique in that, although a relatively small project (2,000 acre-ft ($2.5 \times 10^6 \text{ m}^3$) per year versus Governor Brown's goal of 400,000 acre-ft ($500 \times 10^6 \text{ m}^3$) per year of reuse by 1985), it is a multi-agency project, being implemented at the local level by retail water purveyors.

ST. PETE, FLORIDA IMPLEMENTS REUSE

William A. Duynslager
Public Utilities Director
1635 Third Avenue North
St. Petersburg, FL 33713

St. Petersburg, Florida has implemented a water reuse program to conserve its water, financial, and energy resources. It would appear that this program could be utilized where water supplies are limited or distant, advanced wastewater treatment (AWT) is required, or a combination of the two. Cost savings to St. Petersburg is in the millions of dollars per year.

WASTEWATER TREATMENT CONCEPT

The City's four regional wastewater treatment plants have been or are being converted to recycling systems. A complete-mix secondary treatment plant removes 90 percent of the biochemical oxygen demand (BOD) and suspended solids. The treatment process includes grit removal, aeration, and clarification. A tertiary treatment unit is added using multi-media filtration, flash chlorination, and twelve hour retention storage with added chlorination when needed for quality control. The effluent water, which still contains nutrients but is odorless and as clear as drinking water, is distributed through an 82 mile dual water system for spray irrigation and/or industrial uses.

Public health is a natural concern. Dr. Flora Mae Wellings, the Director of Florida's Epidemiology Research Center of the State Department of Health and Rehabilitative Services, has lead the research and monitoring of the virus control efforts. Her research has determined that the process will reduce the virus to a non-detectable level. Therefore, the effluent can be safely used for irrigation of public parks, golf courses, school yards and residential lawns. The City requires that irrigation utilization be done only through mechanical systems and not via garden hoses. This policy will minimize the accidental ingestion of effluent by drinking from hoses.

WATER SUPPLY CONSIDERATIONS

Any discussion of a dual water supply program must consider all aspects of resource management. St. Petersburg represents the southern third of Pinellas County located midway down the western coast of the Florida peninsula. The County itself is shaped much like the Florida peninsula as it forms that portion of land between Tampa Bay and the Gulf of Mexico. It is nearly surrounded by salt water.

Local water supplies were exhausted in 1928 when salt water intrusion occurred which resulted from local wells pumping two million gallons of water per day. An engineering study concluded that the most cost effective source of potable water would be by wellfield development in northwest Hillsborough County 30 miles away. That portion of the neighboring county was largely undeveloped. The first wellfield was expanded in the 1950's and a second one constructed approximately five miles further away in the early 1960's. A third wellfield located another four miles away in Pasco County in the early 1970's and the latest wellfield was located yet another 10 miles away. Water from the last wellfield is pumped 49 miles to the City for distribution. The water supply facilities, i.e., wellfields, pipelines, water treatment plant, pumping stations, are designed for 82 mgd. The water supply required for 2020 is projected to be 95 mgd or 13 mgd above the design of current facilities. Modifications to the facilities can be constructed for \$10.5 million to force the additional water through the pipelines or a new pipeline with facilities can be constructed for about \$50 million.

A similar water supply cost faces any city which must import high quality water over long distances or which has an adequate supply of water but of low quality. Expensive water does not have to be used for irrigation or many industrial purposes. When evaluating a dual water supply system, the savings to the potable system must be considered.

It is difficult to place a dollar value on the politics developed when one area exports water from another. The undeveloped wellfield areas of the two adjacent counties began development in the 1960's. The question of water rights arose and has continued to develop into political disagreements including several legal questions taken to court.

It is obvious that a desirable solution to the water supply problems facing this city would reduce the ultimate potable water demand upon the system, thus eliminating considerable expansion capital costs; minimize water imported from distant sources because

of pumping costs; and minimize imported water because of political difficulties with adjacent county governments. Unfortunately, these benefits can barely be recognized by EPA in their cost effective analysis of the wastewater program. This is clearly a weakness in the EPA evaluation process. The desired goals can be realized if some of the potable water is reused to meet a portion of the total water requirements of the city. In other words, once used wastewater need not be wasted water.

PUBLIC AWARENESS

St. Petersburg is a summer sort of town where it's easy to take the quality of life for granted. Nowhere in the City can you be more than four miles from open water. It is a city of sunshine, beaches, boating and fishing. The St. Petersburg Evening Independent newspaper published an ad on September 1, 1910 declaring that the paper would be free at newstands any day the sun doesn't shine on the City. This infrequent observation of the City's marvelous climate has occurred only 259 times since the practice was begun.

Development around Tampa Bay was rapid in the late 1940's and throughout the 1950's and 1960's. Pollution controls were sometimes ranked lower in priority than industrial expansion. EPA rated Tampa Bay in 1971 as having the most polluted shoreline in the nation. This rating, plus the area's dependence upon tourism, stimulated the City to authorize a research and engineering project that would involve all levels of government and experts from many disciplines. The public was aware.

Research aimed at having no wastewater discharge into the bay was soon underway. The reuse program was recommended if virus could be controlled. Citizens and civic groups expected improvements. Virus research data was scarce, but ongoing research indicated the possibility that adding multi-media filters to a well treated secondary effluent would work. The City developed a twenty minute slide which was presented with a coordinated taped recording for explaining the program to civic groups and schools. The public information program was kept in low profile but used constantly during the development years. The public became informed and supported the reuse concept. Further attention was directed to the concept when the first wastewater treatment plant was reconstructed to supply safe effluent to the dual water system in 1976 when the National Society of Professional Engineers selected the plant as one of the ten Outstanding Engineering Achievements of the Year. Considerable segments of the public are now waiting for the distribution system to be extended to their areas of the City to allow their utilization of the water.

BENEFIT/COST CONSIDERATIONS

Actual operational benefit cost data is now available. The options to the City were to reuse the wastewater with deep well storage of unused effluent during wet weather periods or AWT, assuming the necessary federal and state discharge permits could be obtained for the Bay. It is doubtful that the State would issue such permits, thus requiring an extremely expensive Gulf outfall. The reuse concept is compared to an AWT alternative, since this would be the most likely alternative available to other locations around the country.

Florida wastewater treatment plant capital costs are compared in the following pages. St. Petersburg has four regional plants ranging in size from 6.4 mgd to 20.0 mgd for average daily flow. Florida Department of Environmental Regulation (FDER) guidelines for cost estimating project that standard secondary treatment plants to meet the specific 62.4 mgd requirement could cost \$46.4 million. AWT plants would cost \$78.1 million. However, the dual water system approach required \$6.6 million for filters, \$19.7 million for 82 miles of the dual distribution pipelines, \$5.3 million for irrigation systems for public parks and schools plus \$4.9 million for deep well storage. Therefore, the total reuse concept totals \$82.9 million, including the \$5.3 million irrigation systems as compared to the AWT \$78.1 million excluding any irrigation systems regardless of the source of irrigation water. The capital costs based upon the reuse concept including irrigation systems and deep well storage is only six percent higher than the AWT alternative.

Operation and Maintenance (O+M) costs for Florida wastewater treatment plants is examined on the following page. Again, the projected O+M costs for secondary and AWT plants was obtained from the FDER guidelines and reflect a secondary O+M cost of \$2.272 million per year compared to AWT at \$6.032 million per year for the 62.4 mgd combined plant capacity. The O+M costs strictly associated with the reuse or dual water system amounts to \$0.683 million per year. Therefore, an O+M savings of \$3.077 million per year is realized by the City. These figures include debt service.

The effluent supplied to the reuse program is clear and odorless with virus at the non-detectable level. It does contain nitrates, phosphates and potassium in the dissolved state. While these substances may upset the digestive system if ingested by man, they are valuable nutrients for plant growth. Actual experience by one golf course customer indicates that an annual savings of

30 tons of fertilizer used on the 60 acres of fairways is expected. This represents a 50 percent savings amounting to \$6,000. These results are based upon an application rate of slightly over one inch of irrigation per week.

The U.S. Soil Conservation Service recommends the annual nutrient application rate specified in the next chart. The typical homeowner would have a \$12.44 fertilizer bill. It is doubtful if many homeowners apply fertilizer at this recommended rate. The nutrients supplied by slightly over one inch per week irrigation is compared to the recommended and reflects \$7.09 worth of value. While this represents about 60 percent of the recommended fertilization, it quite likely would eliminate most homeowners from adding fertilizers to their lawns if the lawns were maintained in the pre-irrigation levels.

A cost comparison for evaluating the reuse-duel water supply system and alternative AWT is presented in the following chart for the total resource management system. The capital cost of the reuse system, including \$5.3 million for irrigation systems, is \$82.9 million with an associated O+M annual cost of \$2.955 million. The alternative AWT program would cost \$78.1 million for wastewater treatment plants, cause \$10.5 million in potable water system construction eliminated by reuse and cause at least \$0.015 million in shallow irrigation wells to be constructed by business or homeowners which would be unnecessary with the reuse option. The capital costs required by the AWT program is \$88.615 million without irrigation systems as compared to \$82.9 million for reuse with irrigation. The reuse savings is over six percent.

O+M annual costs compared the same way plus recognition of the fertilizer benefit of reuse or cost to the AWT option clearly shows the benefit of reuse (\$2.955 million) over AWT (\$6.610 million). It is apparent that reuse deserves a careful and complete analysis when reviewing the total water resource management program.

ENERGY CONSIDERATIONS

Most considerations would have terminated at this point. However, it is appropriate to explore the total program in greater detail. Energy in Florida is mostly obtained from oil. The large difference in O+M costs between the reuse and AWT option is energy and chemicals. Only a minor amount is in labor since the labor force required for the reuse program compares quite evenly with AWT. The O+M costs for the additional potable water system requirements by the AWT option are electrical and chemical. The O+M costs of wells are electrical, and fertilizers are chemicals mostly derived from the petro

industry. Therefore, the majority of the \$3.655 million annual savings is directly or indirectly related to the petro-chemical industry. These costs are projected to increase over the next few years. It can be safely assumed that the cost separation will widen instead of converging. Another reason for reuse.

The St. Petersburg program conserves the water resource, saves substantial amounts of money for its ratepayers, reduces political tensions with neighboring counties and conserves considerable amounts of energy.

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FLORIDA WASTEWATER TREATMENT PLANT

P L A N T	Plant Size (mgd)	CAPITAL COSTS (In millions of dollars)		
		Secon- dary	Reuse	A.W.T.
No. 1	6.4	6.4		10.1
No. 2	16.0	12.0		20.0
No. 3	20.0	14.0		24.0
No. 4	20.0	14.0		24.0
St. Petersburg Costs				
Filters			6.6	
Distribution (82mi)			19.7	
Irrigation (7600 AC.)			5.3	
Deep Wells			4.9	
TOTAL	62.4	46.4	36.5	78.1

TOTAL REUSE = 46.4+36.5=82.9

FLORIDA WASTEWATER TREATMENT PLANT

P L A N T	Plant Size (mgd)	O & M COSTS (In millions of dollars)		
		Secon- dary	Reuse (St. Pete Costs)	A.W.T.
No. 1	6.4	0.374	0.070	0.864
No. 2	16.0	0.584	0.175	1.518
No. 3	20.0	0.657	0.219	1.825
No. 4	20.0	0.657	0.219	1.825
TOTAL	62.4	2.272	0.683	6.032

TOTAL REUSE = 2.272+0.683=2.955

HOUSE LOT FERTILIZER REQUIREMENT

Nutrients Required		Nutrients Supplied
22.4 LB	N	12.70 LB
8.4 LB	P	4.16 LB
8.4 LB	K	9.56 LB
COST \$12.44		\$ 7.09

ST. PETERSBURG RESOURCE MANAGEMENT SYSTEM

	Costs (In Millions of Dollars)			
	Capital		O+M/yr.	
Reuse	82.9		2.955	
A.W.T.		78.100		6.032
Water		10.500		0.500
Private Wells		0.015		0.002
Fertilizer Benefit		0		0.976
Total	82.9	88.615	2.955	6.610

REUSE PLANNING AT EAST BAY MUNICIPAL UTILITY DISTRICT

Donald G. Larkin
Assistant General Manager and Chief Engineer
East Bay Municipal Utility District

It is a pleasure to speak at this Symposium on Water Reclamation on a topic which will probably offer the water industry many challenges in the coming years. The need for water to meet the projected demand in and around the San Francisco Bay area and, for that matter, the entire State of California vividly shows the importance for water resource managers to come up with methods that can make water reclamation technically possible and economically acceptable. In this symposium, the various water reclamation projects in California, both local and regional, are being discussed. My objective today is to tell you the status of the studies on water reclamation at the East Bay Municipal Utility District -- a "subregional" project.

The East Bay Municipal Utility District is a public corporation created by a vote of the people in 1924 to provide a municipal water supply on the east side of San Francisco Bay. The City of Oakland is the largest of the 15 cities within the District, which also includes many unincorporated towns in Alameda and Contra Costa Counties. The population of the District's 300 square mile service area is a little over one million -- about one-quarter of the San Francisco Bay area population.

The District obtains about 95 percent of its water from its Mokelumne River Project located in the foothills of the Central Sierra mountain range, about 100 miles to the east. This project, which is owned by the District, represents an investment by the people of the District of over \$300 million. With existing facilities, the District's water right of 325 million gallons of water a day can be delivered by three aqueducts to the East Bay area. However, the present safe yield of the project is only about 200 MGD. Water use in the service area is roughly divided as Residential - 50 percent, Commercial and public authority - 25 percent, and Industrial - 25 percent.

In addition to its water service function, the District also operates a wastewater agency known as Special District No. 1 (SD 1). SD 1 is the largest wastewater discharger on the east side of San Francisco Bay, with an average flow of about 90 million gallons a day. SD 1 secondary treatment facilities were just completed last year. Full treatment at the secondary level is now being accomplished by means of an oxygen-activated sludge system.

District water planning engineers had periodically investigated the feasibility of water reclamation starting back in 1966. In fact, three technical feasibility reports were prepared for the District from the period 1966 to 1973. It has been District intent that reclaimed water should be considered as another potential source of water to be implemented when proven economically feasible. In 1974, when the District was offered a \$60 million grant to construct secondary treatment works, a condition of the grant was that the District study the feasibility of reclamation, and to develop a Project Report, a Revenue Program, and an EIR on the most feasible water reclamation program. (EIR means Environmental Impact Report, a requirement of California's Environmental Quality Act (CEQA)).

The three reports in the feasibility study are nearly finished.

The District's study of reclaimed water has been concerned with three basic subjects -- the source of wastewater, the market, and the distribution system linking them. The study has been complicated by the existence of eight wastewater treatment agencies in our water service area. All of these agencies are located along the San Francisco Bay shoreline, discharging into the Bay. All of these wastewater agencies now provide secondary treatment, and all were involved with the State and EPA in developing the new secondary treatment facilities. These agencies were contacted to obtain samples for quality data and flow rates. Potential sites for reclamation facilities were determined by researching the available land parcels near the existing treatment facilities.

The marketing analysis of potential users of reclaimed water was a key element in the Study. The study team which made the market study was composed of District engineers and an industrial water expert with the consulting firm of CH2M-Hill. When the study team first began deciding which users were the best candidates for using reclaimed water, it became evident that large non-potable type users should be approached first, especially those located close to one of the wastewater treatment plants. It was reasoned that we should develop a project with the larger customers to take advantage of the economy of scale and with every effort to obtain the least length of separate pipe to deliver the reclaimed water from the wastewater plant to the customer's site.

As some of you may know, all reclamation projects in California must comply with the State Department of Health Regulations. These regulations prohibit any form of ingestion or contact, or cross connection with the drinking water supply. This meant that in our proposed project plans, we had to design a new and separate distribution system to the selected users.

After research of our major customers for location and amount of non-potable use, we settled on 10 industrial users and 22 landscape irrigators. As one would expect, there was no overwhelming demand for reclaimed water, especially since the present water supply is of such high quality. Staff engineers visited representatives of these plants to sell them the idea of assisting the staff in a study that accurately developed the most feasible reclamation program and to point out all technical, institutional and economic issues pertaining to implementing a project. At the opening meeting with each user, we explained that as their water supplier, the District's primary objective was to provide water service to suit their needs. It was explained that it was not the District's intention to deliver any water that created undue hardship on them, or did not meet their facility's process requirements. We explained that it was not our plan to develop a project by District staff alone, but to involve their staff in our technical investigations. We found that the technical staffs of most of the industries were familiar with their water process requirements and were most helpful to the District staff in developing the water quality criteria and fluctuating flow patterns which their plant experienced daily, weekly, and seasonally. Our marketing work, which included several meetings and phone calls with each user's technical staff, took almost two years to complete. Two reports were done on the marketing work -- one covering the industrial reuse market, and the other discussing landscape irrigation. These reports pointed out that some industries' futures were uncertain, and they were trying to

sell out; others were stable and showed potential to use reclaimed water. The three refineries in the service area all indicated the greatest stability and, therefore, good potential markets.

One of the important considerations in choosing a feasible market is rate of demand for water. The peak to average demand is an important factor in sizing the treatment and distribution facilities. The system facilities need to deliver the peak flow, so the higher the ratio the greater the system will be oversized compared to average flow. Industrial water users generally have low peak to average demand ratios, with refineries at 2:1. On the other hand, the landscape irrigation users showed a much higher fluctuating use. A typical peak to average use was 12:1. Their consumption is typically high in the summer and almost zero in the winter. As a result, a water reclamation system built solely for this situation would be operational only about half the year, with large peaking uses in July.

Once the market and the wastewater sources were identified, the study team was prepared to develop preliminary distribution systems and eventually evaluate the most acceptable plan.

Education and input of the public also was an early concern of the District. As mentioned earlier, this study was to also include a Revenue Program and an EIR, each prepared in compliance with EPA grant regulations. The original grant contract for the study specified that the District must, at a minimum, involve the public by holding public hearings on the Draft EIR. In keeping with the spirit of involving the public and the District's desire to obtain outside ideas and concerns early in the study, a Citizens Advisory Committee was formed to advise on the development of the EIR. The Citizens Committee, comprising 25 people from throughout the service area and representing all types of organizations -- environmental, developer, engineer, public advocate groups, etc., met every two weeks with the study team while project report and EIR were being prepared. The Committee assisted the staff in selecting alternatives to be retained for detailed analysis. With ten industrial users, 22 irrigation users, and eight different wastewater sources, one can imagine the numerous permutations and possible combinations that would constitute different reclamation projects.

After months of study and discussion, a reclamation project was chosen which actually consisted of three separate projects. One project was composed of only landscape irrigation users served from the San Leandro City Water Pollution Control Plant. The users of the project are all close to the treatment plant. This project would deliver about 1 MGD on an average annual basis. A carryover storage reservoir and piping system were designed to deliver the expected peaking demand of the users. Although cumulative consumption was assumed in sizing facilities, a way to reduce the overall size of the facilities would be to assume that the users would use water on a pre-determined schedule.

The other two projects are not mutually exclusive, but are actually two alternatives to serve Chevron's Richmond Refinery and a few other minor users. The Refinery's cooling tower makeup demand of 5 MGD comprises most of the potential reclaimed water use. The two projects assume different wastewater sources, one from the District's SD 1 and the second from the City of San Pablo's wastewater plant. Assuming that the District decides to construct one of these projects, further testing of the effluent of the wastewater sources and evalua-

tion of the reliability of future wastewater quantity and quality from the sources will be made before a decision is reached on which of the two projects to implement.

The tertiary treatment processes for the refinery project are designed to achieve the following quality requirements which are specifically aimed at use for the cooling tower makeup:

Orthophosphate as P	<2 mg/L
NH ₄ ⁺ - N	<5 mg/L
BOD, SS	<20 mg/L

Again, before any project would be constructed, the District would pilot test the effects of the treated wastewater on a test cooling tower and then involve the refinery staff in monitoring tower effects. Throughout the course of our discussion with Chevron's representatives, we assured them that a physical testing program would precede any design and implementation of a project to serve treated wastewater to them.

The economics of the reclamation project are not good. The draft project report concludes that an increment of 7 MGD of the present fresh water supply is considerably less costly than the approximately 7 MGD identified as the maximum possible flow from the reclaimed water projects. Some of the Citizens Committee members accept this conclusion, and others believe that water reclamation should be considered on its own merits. They believe that a project should be undertaken based on the need for experience in reclaiming water and on the opinion that it is the responsibility of the water purveyor to maximize its water supply resources regardless of economic cost. Some interesting comments have continued to flow to us from the committee, and with the conclusion of the draft Revenue Program Report, more comments are expected.

The Revenue Program Report sets forth both a proposed financial plan and a revenue program. It was prepared by CH2M-Hill, the same firm preparing the EIR. An important question in the financial plan is who should be identified as beneficiaries of the water reclamation project. All users in the water service area were considered to be benefiting from the reclaimed water project, so that in developing that Revenue model, each water customer was assumed to pay a proportionate share of the project construction and operation costs. Preliminary estimates of monthly charges range from 21 cents to 45 cents for a residential customer, and \$18 to \$40 for an industrial customer, assuming grant funding for Step 2 and 3 projects. Without grant funding, costs will increase by as much as threefold.

To obtain perspective on these costs of reclaimed water, the monthly cost of water to District residential customers (based on May 1, 1979 rate schedule) will average about \$4.45 for a small user, and \$11.95 for a large user. Industrial users will range from about \$600 per month to about \$400,000 per month under the new rate schedule.

Although the cost of reclaimed water in all cases is a small additional fraction on the water bill, the Revenue Report's preliminary conclusion is that the customers cannot afford the increased cost because the net effective income in the service area has declined in relation to the buying power of the dollar. The report points out that any increase in water rates for a reclamation project that merely replaces water that the District already has is unjustified.

It is difficult to make a comprehensive report on a study that has not been completed. The District's facility report of three elements -- Project Report, EIR, Revenue Report -- will be thoroughly reviewed and discussed at public hearings and by the District's Board of Directors before finalization and decision on possible programs. However, at the present stage in the study, it is safe to say that an irrigation project may be economically feasible, if another jurisdiction (the City of San Leandro) develops it both financially and technically. The industrial projects identified in the reports are not economically feasible based on conventional analysis. The District now must consider the other effects -- environmental and political -- of a project. These factors could indicate other reasons justifying a water reclamation project -- such as to help promote the technology and experience of producing and distributing another water supply.

THREE CALIFORNIA WATER RECLAMATION
CASE HISTORIES

Richard J. Stenquist
Project Manager
Brown and Caldwell
Walnut Creek, California

Robert M. Hunter
Project Manager
Brown and Caldwell
Walnut Creek, California

Robert L. Mills
Project Manager
Brown and Caldwell
Walnut Creek, California

Myron E. Steele
Deputy Director of Public Works
for Engineering
City of Santa Rosa, California

Water reclamation has been practiced to a limited extent for many years in California. The need for additional supplies in water-short regions of the state was emphasized by the recent two-year drought, and an accelerated interest in reclamation projects has been the result.

Presented in this paper are discussions of three specific projects, each in a different stage of development. The objectives of these discussions are to review the steps leading to the project, to describe the types of reuse practices in each situation, and to recount some of the difficulties faced in implementing the project. It is hoped that this will aid other communities which are contemplating a reuse program of some type.

The three study areas are shown on Figure 1. The Milpitas-North San Jose project is presently in the middle of the planning phase and will supply water to a relatively large number of low-volume industrial and agricultural users. This has been a water-short area for many years, and several reclamation projects are presently being contemplated.

The Napa-American Canyon program, which is about to enter the design phase, will accomplish two objectives: (1) provide irrigation water to a nearby agricultural area, and (2) reduce wastewater management costs by reusing oxidation pond effluent instead of treating it in a physical-chemical algae removal plant and discharging it to the river.

The Santa Rosa, California, effluent irrigation program has been operational for several years and was recently expanded to include reuse of effluent from a second treatment plant. The Santa Rosa program is an alternative to advanced waste treatment, which is required for summer (dry weather) discharge to receiving waters.

MILPITAS-NORTH SAN JOSE WASTEWATER RECLAMATION PROJECT

The Milpitas-North San Jose Wastewater Reclamation Project has been found to be a cost-effective means of supplying reclaimed water to numerous, relatively small water users in northern Santa Clara County, California. Reclaimed water would be obtained from the San Jose/Santa Clara Water Pollution Control Plant and subjected to additional disinfection prior to distribution. Proposed uses of the reclaimed water include agricultural irrigation, landscape irrigation, and industrial reuse.

Background

Santa Clara County is a water-short area. Prior to the mid-1930's, the groundwater basins which underlie the Santa Clara Valley, from which the county obtains most of its water, were severely overdrafted. This overdraft condition caused land subsidence in the San Jose area and in the baylands to the north. The situation was stabilized only with additional in-county surface water development, importation of water via the State Water Project, and construction of facilities that allowed these waters to be used for groundwater recharge.

Comparison of water demand projections with available water supplies indicates that demands in the county will exceed available supplies before 1980. By the year 2020, a minimum annual water supply deficit on the order of 145,000 acre-ft per year is anticipated. In response to the projected deficit, the district has investigated alternative sources of water supply and has selected the San Felipe Division of the federal Central Valley Project as the primary source of supplemental water for the county. The district has also, however, initiated several wastewater reclamation projects in cooperation with the U.S. Bureau of Reclamation, the California State Water Resources Control Board (SWRCB), the California Department of Water Resources (DWR), and local water purveyors. The Milpitas-North San Jose Wastewater Reclamation Project is one of these programs.

Study Area Characteristics

The study area comprises a 36-sq mi section of northeastern Santa Clara Valley at the southeastern limit

of San Francisco Bay. The area is multijurisdictional and includes portions of the incorporated cities of Milpitas, San Jose, and Santa Clara.

Lowland soils with poor or somewhat poor drainage and a seasonal high water table are the predominant soils used for agricultural purposes within the study area. Irrigation with reclaimed water of high mineral content in areas with a seasonal high water table could lead to a saline condition in otherwise normal soils. The need for special management techniques was therefore considered during facilities planning.

Land use in the study area is rapidly changing from predominantly agricultural to residential and industrial. The area contains the "Golden Triangle," a potentially active site for industrial development bounded by two major freeways.

Wastewater Facilities and Characteristics

A knowledge of existing wastewater conveyance facilities was obtained to identify opportunities to separate flows of higher (or lower) quality wastewaters. An analysis of existing treatment facilities was directed toward determining whether additional facilities would be required to reliably produce a high-quality reclaimed water.

Wastewater Facilities. The San Jose/Santa Clara Water Pollution Control Plant (WPCP) provides secondary treatment of wastewater from a large portion of the Santa Clara Valley. Secondary treatment is accomplished by a modified activated sludge process which uses nitrogen and solids from the digesters to control and stabilize the process. Tertiary facilities that will provide for nitrification, filtration, and improved disinfection are presently under construction.

Three major interceptor systems convey wastewater to the San Jose/Santa Clara WPCP. Each interceptor constitutes a separable source of reclaimed water. The largest interceptor system serves the City of San Jose and eastern Santa Clara.

Western Santa Clara and Cupertino contribute flows to the second largest interceptor, a force main from the North Santa Clara Pumping Station (NSCPS). The smallest system conveys flows from the Milpitas Sanitary District to the WPCP. If Milpitas wastewater were selected for reclamation, the Milpitas Wastewater Treatment Plant (WWTP) could be reactivated to treat the flow.

The Milpitas WWTP was abandoned in 1974 in favor of pumping Milpitas flows to the San Jose/Santa Clara WPCP. The WWTP used a "Rapid-Bloc" modification of the

activated-sludge process which was plagued with upsets from the start of operation in 1965. Plant performance was poor during its operation, despite numerous modifications and a stringent industrial source control program.

A functional analysis of the WWTP indicated that most structures are in very good condition, but that much of the equipment would have to be replaced or repaired to reactivate the plant. Extensive modifications would be required to reactivate the secondary treatment facilities.

Wastewater Characteristics. Wastewater characteristics particularly relevant to reuse potential include wastewater volume, mineral content, and trace constituents.

Analysis of wastewater flow data indicated that with the exception of the Milpitas flow, each source could supply water in excess of anticipated reclamation water demand. With the larger sources, constant flow could be extracted for reclamation. With the Milpitas source, the entire flow would probably be utilized and provision for equalization of diurnal and seasonal variations in flow rates would be necessary.

During February through May 1977, data were obtained on mineral quality for the three major interceptor systems tributary to the San Jose/Santa Clara WPCP. Specific conductance of the wastewater streams was continuously monitored to quantify the variability of their mineral content; values of the specific conductance at one-hour intervals are shown on graphs on Figure 2. Recurring diurnal patterns were noted, characterized by high midday peaks and low early-morning values for the combined San Jose/Santa Clara and Milpitas sewers and by early morning peaks and low nighttime values for the SCNPS force main.

Composite samples were analyzed for selected constituents during this study. Results for 24-hr composites are shown in Table 1. The 8 hr of lowest specific conductance for each source were selected, and 8-hr composites were assembled and analyzed. It was noted that while the 8-hr composites had lower concentrations of nearly every constituent, the decrease was small (4 to 8 percent for TDS) and substantially less than the variations between the sources. For both composite samples, the Milpitas flow had significantly lower values for total dissolved solids (TDS), calcium, and chloride; for most other constituents, however, no consistent pattern could be discerned.

Chemical characteristics of the San Jose/Santa Clara WPCP influent and effluent streams were evaluated with regard to their potential for reclamation for agricultural irrigation, landscape irrigation, and industrial use. Although the salinity of the wastewaters fell within a problem range for agricultural reuse, good management

practices would allow for irrigation reuse of these waters without substantial reductions in crop yields. These management practices include standard agricultural techniques applied in the control of saline and saline-alkali soils. Maintaining good drainage and permeability conditions is central to this control. Leaching fractions of 20 to 30 percent or more may be required. Furthermore, occasional gypsum applications of 1 to 5 tons per acre may be necessary to maintain adequate permeability, particularly in fine-textured soils. Severe high water table conditions could necessitate the installation of artificial drainage for salinity control. Additional techniques may be beneficial, such as (1) preplant irrigation with a better mineral quality freshwater source; (2) double-row seed placement on the shoulders, rather than the centers, of raised beds; (3) use of sloping beds with seed rows placed on the sloping side, just above the water line; and (4) alternate furrow irrigation.

Wastewater boron levels ranged from 0.5 to 0.8 mg/l and were high enough to affect sensitive crops, particularly apricots, cherries, pears, and walnuts. In each of the samples, all trace element concentrations fell far below the recommended maximum allowable concentrations for use up to 20 yr on fine-textured soils of pH 6.5 to 8. It should be noted that this may be the only appropriate criterion since the land will be changed to other uses in less than 20 yr. The permissible concentrations under conditions in the study area is considerably higher because of these soils' capacity for immobilizing trace elements. The trace element levels of the wastewaters sampled may be acceptable for long-term crop irrigation.

Because experience in other areas of California has indicated that a primary market for reclaimed water is as cooling tower makeup, it was reasonable to evaluate the alternative sources with regard to their potential as cooling water supplies.

Review of the mineral characteristics of the potential sources of reclaimed water indicated that, in general, the streams were of comparable mineral quality in regard to their use as an industrial cooling water supply. The SCNPS stream contained higher concentrations of minerals, particularly calcium and sulfate and, thus, was less promising as a reclaimed water source. The Milpitas flow was the least mineralized of the streams monitored, but its limited volume is a serious constraint to its use. Some benefit would accrue from the selection of a nitrified San Jose/Santa Clara WPCP effluent as a source of reclaimed water. Lower ammonia concentrations in the reclaimed water would minimize corrosion problems and would reduce chemical requirements for phosphate reduction. The biological nitrification process also consumes alkalinity, and a reclaimed water with lower alkalinity would also reduce chemical consumption for scale control.

Wastewater Reclamation Potential

A market assessment was conducted during the facilities planning study to identify potential users and uses of reclaimed water in and near the study area. The primary types of potential demand were found to be agricultural irrigation, landscape irrigation, and industrial use.

The estimated landscape irrigation demand exceeded 3,000 acre-ft per year in 1976 and is projected to increase to 5,000 acre-ft by the year 2000. Major factors determining the potential of the landscape market include the size of the demand at a given site, the willingness of the user to use reclaimed water, the feasibility of installing dual piping systems, and the proximity of the user to other users. Golf courses, parks, schools located near parks, and industrial parks appear to be strong landscape irrigation markets.

Demand for water for agricultural reuse was approximately 8,000 acre-ft per year and is anticipated to drop to less than 3,000 acre-ft by the year 2000. The feasibility of using reclaimed water for agricultural irrigation depends on the willingness of farmers to accept reclaimed water, the impact of the water on crop yields and on soils, the ease of converting irrigation systems, compliance with regulatory agency standards for reclaimed water use on crops, and the projected tenure of the farmer. A detailed questionnaire was sent to potential users and follow-up interviews were conducted. The findings showed a significant interest by farmers in using reclaimed water.

In order to determine the size and nature of the current industrial demand for reclaimed water, a preliminary market assessment was conducted in and near the study area. Selected potential reclaimed water users were then contacted to determine detailed information on specific water uses of the industry, the water quality requirements of each use, the amount and timing of water demand, and the piping required on the property to supply reclaimed water.

Recommended Plan

Engineering, economic and environmental factors were considered in development of a recommended plan for wastewater reuse. Distribution system alternatives as well as treatment process alternatives were considered.

In order to determine the areal extent of the distribution system, an incremental unit cost criterion was developed. A branch of the distribution system was extended to connect additional potential reclaimed water users until the incremental cost of treating and distributing reclaimed water to the last user on the line exceeded the

cost of developing new raw water supplies, approximately \$200 per acre-ft. Thus, marginal cost criteria were used to optimize the areal extent of the reclaimed water distribution system.

Disinfection (chlorine contact) was determined to be the only additional treatment process required to comply with California State Department of Health Services (DHS) regulations. Although not providing phosphorus removal would require that industrial users soften reclaimed water prior to its use for cooling tower makeup, limited contacts with potential industrial users indicated that softening of cooling tower makeup is a common practice in the study area. If further discussions with potential industrial users confirm this conclusion, then the cost advantage of phosphorus removal to these users would be minimal. If phosphorus removal is necessary, lime addition would be used, which would probably also provide the additional disinfection required.

The recommended Stage 1 project would serve agricultural, landscape irrigation and industrial demands. Average annual demands at the end of Stage 1 (the year 1990) will be as follows:

Agricultural landscape	- 3,280 acre-ft/yr
Irrigation	- 2,310 acre-ft/yr
Industrial reuse	- 2,150 acre-ft/yr
Total	7,740 acre-ft/yr

The physical components of the recommended plan include interstage pumping facilities, chlorination facilities and a chlorine contact tank, a ground-level storage basin, a reclaimed water pumping station, an elevated storage tank, and a reclaimed water distribution system. A schematic diagram of the facilities is presented on Figure 3, and a layout of proposed treatment, storage, and pumping facilities is shown on Figure 4. Preliminary design data for the project are presented in Table 2.

Project Facilities. Chlorinated effluent will be obtained from the tertiary filter facilities building of the San Jose/Santa Clara WPCP. An interstage, low-lift pumping station will be constructed adjoined to the tertiary filter facilities building to lift the effluent into a gravity pipeline for conveyance to downstream treatment facilities.

Chlorinated effluent will be withdrawn from the chlorine contact tank effluent channel prior to dechlorination to reduce the chlorine consumption and operating costs of wastewater reclamation. Experience with other nitrified, filtered wastewaters has indicated that a chlorine dose of 20 mg/l or greater is required to produce an effluent coliform concentration of 2.2 MPN/100 ml with about one hour of chlorine contact. Because the San Jose/Santa Clara WPCP chlorine contact tanks will provide only one-half hour of contact at peak flow rates,

it is probable that little or no chlorine addition to the reclaimed water stream will be necessary to assure the maintenance of a chlorine residual throughout the reclamation chlorine contact tank.

Additional disinfection of the reclaimed water will occur in a reclamation chlorine contact tank that will provide an additional one and one-half hours of chlorine contact at peak day demands. Facilities will be available to automatically meter, diffuse, and mix additional chlorine solution into the reclaimed water in the event that the chlorine residual in the tank falls below a preset level. Standby equipment and alarms will also be provided in accordance with DHS reclamation criteria. Facilities for effluent sampling will be provided and analyses will be accomplished in the San Jose/Santa Clara WPCP laboratory.

Disinfected reclaimed water will flow by gravity to an earthen storage basin lined with an impermeable membrane. This basin will allow withdrawal and disinfection of San Jose/Santa Clara effluent at a constant rate (peak day demand rate) and distribution of the reclaimed water at a varying rate (peak hourly demand rate). The storage basin will thus be sized to equalize daily demands.

Elevated storage will be provided in the distribution system that will allow the system to be pressurized with constant speed pumps. The volume of elevated storage will be minimal, as most of the equalizing storage will be provided on the ground. As the system is now planned, users below 60 ft elev (MSL) can be served without need for a second pressure zone. The proposed location of the elevated storage tank was chosen to be near a significant percentage of the system's peak demands and to minimize the height and cost of the structure.

A high-lift reclaimed water pumping station will pressurize the distribution system. The multiple constant-speed pumps in the pumping station will be controlled by pressure sensors located downstream from the pumps and by level sensors in the elevated storage tank.

The reclaimed water pumping station will be sized to supply peak hourly demands with the largest pump out of service. The pumping station will be equipped with surge relief valves and other devices to minimize problems caused by hydraulic transients (water hammer). The decision as to whether or not standby power generating facilities are necessary will be based on the reliability requirements of users.

The reclaimed water distribution system will consist of transmission pipelines located along an east-west roadway with pipeline branches extended in generally north-south alignments along roads and other public rights-of-way into the study area. The distribution system pipelines were sized to accommodate concurrent peak hourly flows with the hydraulic gradient along each line varying from 100 psig

(230 ft) to 50 psig (115 ft) above the pipeline. The resulting pipeline hydraulic gradients were generally higher than those commonly used in water works practice (e.g., 1-3 ft per 1,000 ft).

During design of the system, detailed reevaluation of the hydraulics of the distribution system will be made using actual user demands. It has been recommended that acceptability to users of the assumed pressure range (50 to 100 psig), the impact of reduced hydraulic gradients on system operation and flexibility, and the impact of demand variations on hydraulic transients be analyzed in detail. Trade-offs among such factors as the height of elevated storage, the flexibility of the system to accommodate different demands in the future, and the increases in system reliability that would be accomplished by adding loops to the system should also be analyzed.

Estimated Project Costs. The estimated capital cost of the recommended stage 1 project is \$14,490,000, based on 1978 price levels. Operation and maintenance cost of the Stage 1 project will average \$340,000 annually.

Present Status

A project report summarizing the findings of the facilities planning studies was submitted to local water purveyors and the SWRCB Division of Water Quality in July 1978. Because the service area of the project includes portions of the cities of Milpitas, San Jose and Santa Clara, implementation of the project will be accomplished by those entities.

The city staffs have drawn up a joint powers agreement among the cities, with the City of San Jose as the lead agency, to provide a formal basis for project implementation and cost sharing. Current plans call for initiation of the process of obtaining firm commitments from users of reclaimed water and establishment of institutional and financial arrangements among project participants by mid-1979. During the process of obtaining firm user commitments, a number of issues that could not be resolved without more detailed knowledge of the requirements of specific users will be studied further. Among these issues are the following:

- Development of a reclaimed water pricing policy that would encourage reuse but allow recovery of the local costs of the project. The pricing policy must take into account existing price differentials between water used for agricultural irrigation versus other uses.
- Development of acceptable site-specific management practices for irrigation with a more mineralized reclaimed water.

- Development of design criteria (e.g., acceptable pressure variation at the point of use, need for an uninterruptable source of water, anticipated demand rate peaking factors) for the reclaimed water distribution system.
- Detailed hydraulic analysis of the proposed reclaimed water distribution system to maximize system reliability, minimize hydraulic transients (water hammer), and minimize total annual costs of distribution.
- Determination of the economic feasibility of individual users constructing and operating on-site reclaimed water treatment (e.g., ion exchange of cooling tower makeup) and/or distribution facilities.

NAPA-AMERICAN CANYON WASTEWATER REUSE PROGRAM

Effort has been under way for several years to implement a wastewater reclamation project in the lower Napa Valley, using effluent from Napa and American Canyon to provide water for agricultural irrigation, particularly in the Carneros area southwest of Napa. Napa Sanitation District (NSD) and American Canyon County Water District (ACCWD) are presently undertaking the Napa-American Canyon Wastewater Reuse Program. Major objectives of the project are as follows:

- Provide a source of irrigation water for vineyards and other agricultural uses such as pasture, orchards, hay and silage crops. Present sources of surface water and groundwater supplies are inadequate to allow the full potential of area agriculture to be realized.
- Eliminate or reduce discharge of wastewater effluent to the Napa River, particularly during summertime, low-flow conditions.
- Reduce NSD and ACCWD wastewater management costs by reusing secondary (oxidation pond) effluent. The new NSD/ACCWD joint tertiary facility, which was constructed to remove algae from oxidation pond effluent prior to discharge to the river, has high chemical, power, and labor costs. Although the quality of oxidation pond effluent is inadequate for river discharge, it is sufficient for many reuse purposes.

Background

Wastewater reuse planning in the lower Napa Valley began in 1972 when the Napa County Flood Control and Water

Conservation District (NCFWCWD) commissioned a study on the agricultural water supply potential for Napa County. Included in this study was a project to supply the Carneros area with fully treated reclaimed wastewater from NSD's advanced treatment facility. The Carneros area covers approximately 7,000 acres. This project would have served 1,400 acres of vineyards with water for frost protection, heat protection and irrigation.

In 1976 the County Board of Supervisors received a petition from 60 families and corporations representing 5,000 acres of agricultural land, requesting that the board authorize the initiation of studies to evaluate a potential larger service area and other agricultural uses. The board subsequently authorized the flood control district to undertake the study.

The subsequent report, entitled "Carneros Area Agricultural Water Supply Study," indicated that the irrigation water supply available in the Carneros area is inadequate. At the present time very little of the area is used for irrigated agriculture. The report indicated that the economy of the region would benefit significantly if an economical source of irrigation water could be found. In 1977 the engineering consulting firm of Brown and Caldwell was commissioned by NSD and ACCWD to undertake a facilities planning study of the potential for wastewater reuse in the lower Napa Valley, leading to the implementation of a reuse project. The selected study area is shown on Figure 5 and consists of approximately 67 sq mi; dominant land uses consist of dry-farmed grassland, vineyards, and lesser acreages devoted to orchards and landscaped areas. Also shown on Figure 5 are the NSD and ACCWD service areas and the boundaries of the Carneros Water District (CWD), an entity which was formed in 1978 for the purpose of purchasing and distributing reclaimed water in the Carneros area.

Study Area Characteristics

The study area consists of Napa River's lower alluvial plain and estuary plus portions of the adjacent hills. The area is located to the south of the main part of Napa Valley's famous wine country. As wine production has risen in recent years, however, an increasing acreage of land in the southern part of the county has been converted to vineyards. (Figures 6 and 7).

Soils are generally nonsaline with moderate to heavy surface texture and a subsoil dominated by clay. The clay subsoil effectively separates the surface from underlying groundwater resources. Soils near the Napa River (Reyes classification) are unsuitable for irrigation with reclaimed water, and some of the other soils might require special management techniques.

Napa River is the major surface water stream in the study area and is subject to considerable tidal action from San Pablo Bay. Large areas of former marshland exist in

the study area as well as several rare and endangered plant and wildlife species. Major land uses include dry-farmed grassland and vineyards in the basically rural setting of this lower valley area where agriculture is a major industry. Most of the future population growth is expected to occur in the American Canyon area. Industrial development may occur along the Napa American Canyon highway.

Wastewater Treatment Facilities and Characteristics

Presented below are brief descriptions of the existing Napa Sanitation District (NSD) and American Canyon County Water District (ACCWD) treatment facilities, the joint physical-chemical treatment plant, and the quality characteristics of NSD and ACCWD wastewaters.

American Canyon County Water District. Treatment of ACCWD wastewater flow is provided by a series of four oxidation ponds. The four ponds are operated in series and are designed to have the first pond operate in a facultative mode with the remaining three ponds aerobic. The plant is laid out to be expanded in stages with the ultimate design flow four times that of the existing plant (0.5 mgd). Using the ponds for short-term storage, effluent is pumped to NSD pond No. 1 several hours each day. Although chlorination facilities are available, disinfection is not normally provided when effluent is transferred to the NSD ponds. Previously, effluent was discharged to the Napa River.

Napa Sanitation District. NSD presently operates an 8.0-mgd trickling filtration plant at Imola Avenue just east of the Napa River. In addition, NSD has 340 acres of oxidation ponds approximately four miles south, adjacent to the Napa River and north of the county airport. The four NSD oxidation ponds are operated in a series mode. Previously, effluent was chlorinated and discharged to the Napa River. Maximum pond depth is 8.0 ft and minimum depth is about 3.5 ft, meaning that approximately 1,500 acre-ft of storage is available within the ponds.

Joint Wastewater Treatment Plant. The joint treatment plant located at the NSD pond site provides removal of algae through coagulation-flocculation-sedimentation using alum or lime as the coagulant. This is followed by dual-media filtration for effluent polishing, chlorine disinfection, and dechlorination with sulfur dioxide. The plant receives wastewater from NSD pond No. 4 and discharges effluent to the Napa River. The facility was placed in operation in 1978.

Wastewater Characteristics. Effluent from the Napa Sanitation District and American Canyon County Water District oxidation ponds is considered to be a well-stabilized secondary effluent low in oxygen demanding materials but high in suspended solids due to algae.

Effluent from the new tertiary facility will have very low concentrations of the parameters normally used as indicators of pollution: BOD₅, suspended solids, and coliform organisms. Effluent may be diverted for reuse either after oxidation pond treatment or after tertiary treatment as shown on Figure 8. Although tertiary effluent will possess a much higher quality, use of the tertiary facility will result in very high operating costs. Reuse of secondary (oxidation pond) effluent, whose quality is adequate for many reuse purposes, will result in significant cost savings to NSD and ACCWD.

A wastewater sampling and analysis program undertaken in conjunction with the facilities plan showed that boron levels in NSD and ACCWD effluent vary seasonally from about 0.4 to 1.2 mg/l. Boron can have an adverse effect on grapes. Sodium chloride levels in NSD pond effluent vary from about 300 to over 800 mg/l; this constituent can adversely affect plant growth and cause plugging of clay soils.

Wastewater Reclamation Potential

An important part of the planning process was to determine the technical, social, and institutional feasibility of reclaiming and reusing wastewater effluent.

Area Water Supply and Characteristics. Present water supplies for Napa and American Canyon are of good quality, and the quantity is sufficient to meet municipal demands through the end of this century. Water for agricultural irrigation is limited, however. Small surface reservoirs have been constructed in the Carneros area to capture runoff which is used in large part for drip irrigation systems. Evaluation of potential alternative supplies of irrigation water shows that wastewater reclamation and reuse is more feasible than the three major alternatives: (1) development of a well field in the eastern part of the study area, (2) diversion of imported water from municipal to agricultural use, and (3) construction of a dam and reservoir on Upper Conn Creek north of the study area.

Evaluation of Reclaimed Water Quality for Irrigation. Constituent concentrations in NSD and ACCWD effluent vary seasonally; levels are high in summer due to evaporation in the oxidation ponds and low in winter from infiltration/inflow and precipitation on the oxidation ponds. In evaluating suitability of water for irrigation, attention is paid to four major hazard areas: (1) soil salinity hazard, (2) soil permeability hazard, (3) specific ion toxicities, and (4) miscellaneous hazards. The expected maximum concentrations of sodium chloride and boron (occurring in late summer) result in "increasing" or "severe" problems for three problem areas: salinity (as measured by electrical conductivity), permeability (adjusted sodium adsorption ratio), and specific ion toxicity (sodium, chloride, and boron).

Although boron seems to be widespread in area water and wastewater, much of the sodium chloride appears to be discharged into the NSD sewerage system from one of the local tanneries. Separate treatment for tannery wastewater could reduce sodium chloride concentrations by 40 to 50 percent and would significantly improve the feasibility of reclamation in the area. Modifications to the tannery processes might be another way to reduce the sodium chloride loading.

Institutional Factors. A number of governmental agencies, on the federal, state, and local levels, will have an important influence on the proposed project.

The U.S. Environmental Protection Agency is responsible for disbursement of federal wastewater treatment works construction grant funds, which are expected to constitute a significant fraction of the project's capital cost. EPA is also charged with enforcement of federal water pollution control laws.

The State of California is playing a major role in encouraging water reclamation projects in California. The State Water Resources Control Board, which administers the State Clean Water Grants Program, has developed a "Policy and Action Plan for Reclamation in California." A new Office of Water Recycling has been formed as part of SWRCB and will coordinate and promote water recycling within California. SWRCB also has been delegated authority by EPA to manage federal grant allocations in California, subject to final EPA certification.

The California Regional Water Quality Control Board, San Francisco Bay Region, will set discharge requirements for the reuse project, working within reclamation criteria and guidelines set by the California Department of Health Services. It is anticipated that secondary (oxidation pond) effluent will be satisfactory for most reuse applications in the study area as shown on Table 3.

Reclaimed Water Demand. Potential reclaimed water use in the study area is summarized in Table 4. The four subareas cited were shown on Figure 5. The water volumes shown are approximately equal to the wastewater expected to be produced in the two years shown, 1980 and 2000. Demand in the Carneros subarea may be considered fairly firm, but effort will be required to implement a reuse program involving the full totals shown in Table 4.

Project Alternative Analysis

Alternative projects were developed for the reuse categories shown in Table 3. These were subjected to an initial screening to eliminate those projects which were clearly infeasible. Based on this screening process, several reuse categories were eliminated from further consideration. These include (1) provision of flow

capacity specifically for frost protection spraying of vineyards in the Carneros area, and (2) use of effluent in the Northeast subarea (north of the NSD ponds and east of the Napa River) for landscape irrigation at J. F. Kennedy park and golf course and for industrial reuse at the two local tanneries. A relatively high cost to supply a small volume of water and, for the tanneries, transporting the water a great distance were the major reasons for eliminating alternatives.

Based on information developed during the screening process, the initial set of alternatives was modified prior to final evaluation. An important component of these modifications was inclusion of the use of county airport property for agricultural irrigation. This land is located adjacent to the NSD oxidation ponds and can be secured for use at little cost to the districts.

Based upon a detailed analysis involving cost, environmental and social impacts, flexibility, reliability, and public and agency acceptance, two alternatives were selected for implementation.

Description of Recommended Alternatives

Alternative F3, developed for the Carneros subarea will provide for reuse of 3,700 acre-ft per yr by landowners in the Carneros Water District. The distribution system for Alternative F3, termed "intermediate" in extent, is shown on Figure 9. Design data for the project are shown in Table 5. During winter when reuse demand is low, effluent will be stored in the existing NSD oxidation ponds and in Huichica Reservoir, which will soon be constructed by Sonoma Valley County Sanitation District as part of its reclamation program which is being implemented to the west of the Napa-American Canyon Study Area.

Reclaimed water will be sold by NSD to CWD which, in turn, will deliver it to individual users. Long-term contracts, 10 to 20 yr, will be required of users.

Principal uses are expected to be spray irrigation of pasture and hay crops and drip irrigation of vineyards. Others uses may include irrigation of orchards and highway landscaping and future irrigation of crops such as corn silage which are not presently grown in the region. Reclaimed water may also be used for vineyard frost protection spraying, but capacity specifically for this purpose will not be included in the system design because of high cost. Use of reclaimed water for fire fighting may also be undertaken, but this will require approval of the California Department of Health Services (DHS). Spray irrigation of food crops (including grapes when the fruit has formed) will not be permitted because the water will not meet quality requirements imposed by DHS.

Alternative V5, to be implemented in the Southeast subarea, will provide for reuse of 1,040 acre-ft per yr initially with possible expansion to over 4,000 acre-ft

per yr over several years. Much of the irrigated land will be owned by the county airport. Irrigation of pasture, hay crops, and in the future, such crops as corn silage are the principal uses seen for the Southeast subarea. Project components for Alternative V5 are shown on Figure 9; design data are presented in Table 5.

Possible Future Expansion

Much of initial planning work was necessarily concerned with the Stage 1 projects. Future expansion of these projects and possible development of new reclamation projects should be periodically reviewed, however.

- Carneros Subarea: Limitations on use of state and federal wastewater management grant funds for future capacity limits the amount of planning for future expansion which can be incorporated into the Stage 1 design. Such concepts as additional future storage to meet peak month flow demands and provision of a "loop" by connecting the ends of the Stage 1 pipeline branches does make future expansion practicable, however.

Design peak flow for Carneros area Alternative F3, 16 mgd, is based upon a 12-hr irrigation day during the month of peak demand, July. Use of a 12-hr irrigation day is reasonable for the type of system proposed. Additional land can be irrigated with the recommended system if, for example, an 18-hr or 24-hr irrigation day is used, or if individual land owners stagger their use of water over the day. Cooperation between users and strict adherence to delivery schedules would be required to expand the program in this matter.

- East and Southeast Subareas: In addition to the planned expansion from 1,040 acre-ft per yr to 4,000 acre-ft per yr over the next few years, there is potential for more reuse in this region. Acquisition of more land by NSD and ACCWD, changes in zoning from an industrial to agricultural classification, and possible high profitability of irrigated agriculture may all lead to additional reuse.

Industrial development in this region may also result in increased wastewater reuse. Secondary effluent from the NSD oxidation ponds could be used for landscape irrigation of industrial park grounds. Effluent from the NSD/ACCWD joint tertiary facility can be used for many process water and cooling water applications. Treatment costs could be partially offset by the sale of water to industries.

- Northeast Subarea: Use of reclaimed water for landscape irrigation in the Northeast subarea was eliminated from consideration during the initial

screening of alternatives because of high costs. Future costs increases for alternative sources of water and/or an increase in the acreage of landscaping in the area are two factors which may make such reuse feasible in the future.

Marsh Enhancement: If steps are taken to restore former marshlands to their original state, reclaimed water may play an important role in the process. Guidelines have been set by the San Francisco Bay Regional Water Quality Control Board for implementation of marsh programs. They include a provision for pilot studies to determine the quality of water leaving the marsh and entering surface receiving waters. For this reason, any marsh enhancement program will require a lead time of several years for implementation.

Separate Treatment of Tannery Wastewaters

Chemical analysis of NSD and ACCWD wastewater effluent showed high levels of boron for both and high concentrations of sodium chloride in NSD effluent. As discussed previously, these constituents may hinder successful implementation of a reuse project within the study area. A monitoring program was undertaken to determine the sources of these contaminants. It was found that high levels of boron occurred at most points in the system. Approximately half of the sodium chloride loading, however, could be traced to one of the tanneries located in Napa. As the tannery is located near the NSD Imola Avenue treatment plant, it was concluded that the most cost-effective solution would be provision of separate treatment of the tannery waste at the Imola plant. The treatment scheme chosen comprises primary sedimentation, biological treatment in aerated lagoons and a facultative pond, and evaporation in holding ponds.

Disposal by evaporation would eliminate concern over contaminant concentrations in water discharged to the river or to land. Sufficient treatment to preclude odor generation would be required, but this can be accomplished without the high expenses associated with, for example, either stream discharge or deep well injection. The principal limitation on evaporation is that a limited area of land is available, and this may not be sufficient to provide for evaporation of the entire flow.

Costs and Schedule

The total capital cost for Alternative F3 in the Carneros area, excluding Huichica Reservoir, is estimated at \$8,555,000 at 1980 price levels; the cost for Alternative V5 in the Southeast subarea, is estimated to be \$7,495,000. Separate treatment of the tannery waste is expected to be \$1,335 million. Total cost for the three projects is set at \$17,385,000 at 1980 price levels. Total operation and maintenance costs are expected to be about \$195,000 per year.

The expected effect of the project on wastewater management costs is shown on Table 6. Reuse of 4,740 acre-ft of secondary effluent will reduce tertiary treatment costs by about \$500,000 per year. The net savings in operation and maintenance costs to the district is approximately \$300,000 per year.

The facilities plan for the project has recently been completed, and design is expected to start this summer. It is hoped that the project will be operational for the 1981 irrigation season.

SANTA ROSA EFFLUENT IRRIGATION PROGRAM

The City of Santa Rosa, California, faced with a choice between constructing an advanced waste treatment facility and implementing an agricultural irrigation reuse program, chose the latter course. The result is a program which presently reclaims approximately 4,600 acre-ft of secondary effluent from two treatment plants with the potential to reclaim approximately 7,500 acre-ft.

Background

In 1973, the City of Santa Rosa, California, provided sewerage service for its incorporated area and for adjacent, unincorporated areas of Sonoma County. Between 1962 and 1973, several studies had called for consolidation of treatment and disposal services for Santa Rosa and the nearby cities of Rohnert Park, Cotati, and Sebastopol.

In October 1973, a project report was prepared for the City of Santa Rosa which described a specific program of staged construction to provide a regional treatment facility. The recommended plan called for consolidation of all treatment and disposal facilities at the City of Santa Rosa's Laguna Wastewater Treatment Plant by the end of Stage 1 (1982). Wastewater from Rohnert Park, Cotati, Sebastopol, and Santa Rosa's West College Avenue plant was to be conveyed to the Laguna plant through interceptors constructed during this time period. The capacity of the Laguna plant was to be increased from 2.5 mgd to 15 mgd.

Disposal of effluent from the Laguna plant has been the subject of much study and controversy over the years. Santa Rosa's Laguna plant discharges to the Laguna de Santa Rosa which is tributary to the Russian River. The river boasts many popular summer resorts and vacation homes along its lower reaches, and residents have been pressing for many years to halt upstream wastewater discharges during the summer season. The river is also diverted for water supply in downstream areas.

Originally, the California North Coast Regional Water Quality District, which has authority in California to set treatment and discharge requirements, prohibited discharge to tributaries of the Russian River between May 15 and September 30 and at any other time that the river flow is

less than 1,000 cfs. This prohibition would have required land disposal or storage for a considerable portion of the year.

Subsequent studies, however, indicated that the feasibility of reclaiming all projected quantities of secondary effluent within the watershed was doubtful because of the lack of firm commitments for reuse by local irrigation interests and because of the results of preliminary soils studies in the Laguna flood plain.

Following consideration of this question, the regional board formally modified its position by adopting an amendment to the basin plan, allowing unlimited surface discharge of effluent during summer, provided it is highly treated. Secondary effluent may not be discharged during the May 15-September 30 period. During the October 1-May 14 period, secondary effluent may be discharged at a rate equal to one percent of the Russian River flow when the river flow is less than 1,000 cfs and at a rate equal to 5 percent of the river flow when the river flow is greater than 1,000 cfs.

In recommending a disposal plan for the Laguna plant, the 1973 project report concluded that advanced wastewater treatment, comprising nitrification, denitrification, chemical coagulation, sedimentation, filtration, and chlorination-dechlorination should be used for discharge of effluent to the Laguna De Santa Rosa. It was also concluded that the 5-mgd increase associated with each of Stages 2 and 3 should be disposed of on land after secondary treatment. It was recommended, however, that studies be undertaken to provide further information on the effluent soil loading rates applicable in the Laguna area and on the environmental effects of land disposal. It was recognized that some land application could be undertaken before the end of Stage 1 and that the actual mix of stream discharge at the end of Stage 3 might be different from the 15 mgd and 10 mgd, respectively, cited in the project report.

The city, recognizing the high operation and maintenance costs associated with advanced waste treatment, started to move toward a project which involved secondary treatment (activated sludge) at the Laguna plant, winter discharge of secondary effluent, storage, reuse through agricultural irrigation, and retention of the West College Avenue plant as a reclamation plant to provide nearby farmers with reclaimed water for irrigation. To this end, they authorized a two-year demonstration study to evaluate the feasibility of irrigation with secondary effluent in the Laguna area. This demonstration study led to implementation of the full-scale program.

Study Area Characteristics

The City of Santa Rosa is situated on a low, flat plain sloping to the west-southwest at approximately 10 ft per mile. The climate is typical of central coastal valleys

of California. It is characterized by ocean-moderated temperatures that only occasionally exceed 85 F or drop below 35 F. Fog occurs occasionally throughout the year and is generally dissipated by midmorning. Wind is usually from the south or southwest and generally less than four miles per hour. About 80 percent of the average annual rainfall of 295 inches occurs during the five-month period of November through March.

Length of the growing season for pasture in the valleys and plains in the north coast region is about eight months, lasting from about the beginning of March to the end of October. In the area around the Laguna de Santa Rosa, however, sufficient heat does not exist for maturing a corn crop or for obtaining optimum yield from Sudan grass. Moreover, during most years, cultivation cannot be carried out until early May due to wet soil conditions which are often encountered in the flood plain.

Soil fertility is moderate to high, but nearly all of the soils have a tight clay subsoil 10 to 25 inches below the surface. This restricts downward movement of water and has caused concern over the suitability of the area for effluent reuse.

Groundwater yields in the area are low; during the pilot irrigation study discussed below, part of the water for the control plots had to be hauled in from the city distribution system because of the inability to obtain an adequate supply of well water.

Santa Rosa and its neighboring cities are located on the edge of the San Francisco Bay area and are being subjected to pressures for and against growth. Naturally, the Santa Rosa wastewater reclamation program, with its emphasis on preservation of agriculture, the local dairy industry, and open space, has become a part of the continuing disagreements over the extent and rate of urban growth which is best for the area.

Wastewater Treatment Facilities and Characteristics

The Laguna treatment plant, constructed in 1965, is the largest and newest of the two facilities involved in the reclamation program. The West College Avenue plant, constructed in 1950, scheduled to be abandoned when the Laguna plant was enlarged, has been retained for use as a water reclamation plant for nearby agricultural lands.

Laguna Wastewater Treatment Plant. The Laguna plant provides primary sedimentation and biological oxidation by the activated sludge process. Chlorine disinfection and dechlorination by sulfur dioxide precedes winter stream discharge. The plant is designed and operated to achieve ammonia oxidation to nitrate in the biological oxidation process. For stream discharge, the total coliform level must not exceed 2.2 MPN per 100 ml. For irrigation of silage crops or pasture, a 23-MPN-per-100-ml

coliform limitation is imposed. As noted above, the Stage 1 design flow for the plant is 15 mgd; expansion may be carried out in 5-mgd increments. The plant is laid out so that future expansion may include upgrading of the treatment process to include nitrogen removal by biological denitrification, phosphorus removal through tertiary chemical addition, and filtration.

West College Avenue Sewage Treatment Plant. The West College Avenue plant originally provided primary treatment and biological treatment by trickling filtration with polishing oxidation ponds preceding stream discharge; the design flow is 5 mgd. Recently, however, after nearly 30 years of operation, the trickling filters became clogged and began producing odors. After attempts to clean them out with caustic chemicals proved unsuccessful, the biofilters were abandoned and brush, or cage, aerators were installed in the oxidation ponds to provide additional oxygen there. Effluent from the oxidation ponds is disinfected with chlorine to a 23-MPN-per-100-ml level.

Originally, effluent from the West College Avenue plant was discharged to Santa Rosa Creek, which in turn discharged to the Laguna de Santa Rosa and the Russian River. Santa Rosa Creek is an intermittent stream which had been straightened and improved to carry winter flood flows. During summer, essentially all of the flow in the creek was West College Avenue plant effluent. Landowners adjacent to the creek historically diverted the water for irrigation, and a de facto reclamation program had been implemented.

When the regional board prohibited discharge of effluent to the creek, the city constructed a gravity pipeline (mostly 27-in. diameter) nearly six miles long and 220 acre-ft of storage ponds along the route to serve the landowners directly with reclaimed water. The 200 acre-ft capacity oxidation ponds at the treatment plant provided additional storage capacity. The West College Avenue system is now connected with the Laguna system to provide operational flexibility.

Wastewater Characteristics. Shown on Table 7 are summary data on chemical characteristics of Laguna secondary effluent. For comparison, data are also given for well water and city water, which were used for irrigation of control plots during the irrigation pilot study. These waters can be classified as low sodium hazard and low to high salinity hazard waters. Specifically, city water is an excellent water which will cause no salinity or soil permeability problems. The well water, derived on site, is a medium quality water which may create increasing problems with regard to soil salinity and may cause some chloride and sodium toxicity through foliar (leaf) absorption and possible chlorine toxicity through root absorption. The Laguna plant secondary effluent is also a medium quality irrigation water with respect to salinity hazards and sodium and chloride toxicity; this water may

also cause some problems with regard to soil permeability and may cause some boron toxicity problems for the more sensitive crops.

Effluent Irrigation Study

The pilot effluent irrigation study was carried out during the growing seasons of 1974 and 1975 on a 20-acre parcel of land west of the Laguna Wastewater Treatment Plant (Figures 10 and 11). Various field and forage crops suitable for the climatic conditions of the Santa Rosa plain were raised under different fertilization and irrigation practices. Emphasis was placed on pasture and silage crops of interest to the local dairy industry. Treated effluent and freshwater were used as the sources of water supply. Major objectives of the investigation are summarized below:

- Evaluate design parameters for a full-scale effluent irrigation program.
- Investigate and demonstrate the feasibility of growing field crops with secondary effluent from the Laguna plant.
- Determine if fertilizer benefits can be derived from agricultural reuse of wastewater on crops of interest to local farmers.
- Evaluate the effect of effluent irrigation on the chemical composition, fertility, and structure of soil in the project area.
- Evaluate the effect of effluent irrigation on food value and chemical composition of crops grown on the site.
- Evaluate the effect of effluent irrigation on the number of pathogenic, indicator, and nitrogen-fixing bacteria in the soil.

Crop Yields. Crop yield data for the 1974 season are summarized in Table 8. Crop yields in effluent irrigated plots were lower than control plots (irrigated with well or city water) for all pasture, barley silage, and Sudan grass replicates.

Corn silage replicates had higher dry matter yields in effluent irrigated plots than in plots irrigated with city water. Also, yield variations in corn plots was rather minimal. Dry matter yield in the control plots varied from 3.4 to 4.1 tons per acre with an average yield of 3.8 tons per acre. Six plots irrigated with the effluent had dry matter yields varying from 3.6 to 4.3 tons per acre with an average yield of 4.1 tons per acre. This yield is about 8 percent higher than the average yield obtained from the control plots.

Barley grain replicates which were irrigated with effluent had yields varying from 0.6 to 1.5 tons per acre and an average yield of 1.1 tons of dry matter per acre. The actual grain yield of 1.5 tons per acre which was obtained for two out of three barley plots constitutes an excellent yield for this crop and shows the viability of the use of treated effluent for growing a satisfactory grain crop without application of any commercial fertilizers. Yield results from corn silage plots were also very encouraging because effluent replicates produced a better yield than control plots which were fertilized with 150 lb of nitrogen and 100 lb of phosphorus per acre. The poor yields in the effluent replicates of pasture, Sudan grass and barley silage can be attributed to the need of these shallow-rooted crops for high levels of available nitrogen in the top few inches of the soil. Better yields can be obtained from these crops if supplemental nitrogen is applied to the soil one or more times during the growing season.

Data on crop yields obtained during the 1975 season are presented in Table 9. During this year, commercial fertilizer was applied at various rates to effluent and freshwater plots in order to determine the optimum level of fertilization that will result in maximizing the yield from effluent irrigated plots. The following conclusions can be derived from the data:

- Yield of pasture was improved in effluent plots as a function of the amount of nitrogen fertilizer applied to these plots. At the same level of fertilizer application, effluent irrigated plots had 20 percent higher average yield than freshwater irrigated plots.
- Corn silage yields in effluent irrigated plots were also higher than the yield obtained from freshwater plots. At comparable levels of fertilizer application, the yield in effluent plots was 10 to 30 percent higher than that in freshwater plots. Maximum yield in effluent replicates was obtained with a nitrogen fertilizer application of 50 lb per acre. Additional application of nitrogen fertilizer did not result in any improvement in the crop yield.
- Maximum yield of barley grain crop was also obtained in effluent irrigated plots which received 50 lb of nitrogen per acre. This yield was 20 percent higher than that obtained from the freshwater plots which received 150 lb of nitrogen per acre. Application of 100 or 150 lb of nitrogen per acre did not result in further yield improvement in the effluent irrigated plots.
- Application of nitrogen fertilizer resulted in improved yields in effluent irrigated Sudan plots. However, the highest yield was 25 percent below

the level obtained from freshwater plots in the 1974 growing season which received an equal amount of fertilizer. This reduced yield may have resulted from unfavorable weather conditions which prevailed throughout the 1975 growing season.

Conclusions. The major findings of the effluent irrigation pilot study can be summarized as follows:

- Secondary effluent from the Laguna treatment plant is a water of good quality for irrigation reuse and can be applied to grain or fodder crops on most soils found in the Santa Rosa plain.
- The soil monitoring program showed that effluent can be used continuously for crop irrigation without creating heavy metal toxicity or an excessive salinity condition in the soil.
- The effluent has some nutrient value for crops and will reduce the need for fertilizers on grain or fodder crops.
- The bacterial monitoring program indicated that there were no pathogenic Salmonella or Shigella bacteria in the surface soil mantle in effluent irrigated plots.

Based on the encouraging results of the pilot study, the city implemented a full-scale reuse program.

Implemented Program

Treatment, conveyance, and storage facilities for the Santa Rosa effluent irrigation program are shown on Figure 12 as are the three parcels of land purchased by the city for irrigation reuse. Design data are summarized in Table 10.

On city-owned land, the city is responsible for running the irrigation program. The city receives bids for planting the crops in the spring and again for harvesting in the fall. If necessary because of greater than expected quantities of water, the city can apply excess water to the irrigated fields. This may reduce the yield but causes no significant problems because the land and the crops are city-owned.

Local landowners who receive effluent from the city must sign 10-year contracts. At present, 16 contracts totaling 2,250 acre-ft per year (range: 12 to 340 acre-ft) have been developed for effluent from the Laguna plant. Seventeen contracts totaling 2,350 acre-ft (23 to 300 acre-ft for individual contracts; one user has five separate contract totaling 455 acre-ft) are in effect for the West College Avenue treatment plant. Another 2,000 to 3,000 acre-ft per year can be disposed of on the city's three irrigation systems which total approximately 1,000 acres.

Total capital cost for the Laguna reuse facilities (excludes West College Avenue conveyance and storage facilities) is \$11,000,000. Construction was begun in 1977, and full automation of controls will be completed in 1979.

A layout of the Brown irrigation system is shown on Figure 13. There are 14 primary fields with an average area of 20 acres. Each field is irrigated in turn for approximately six hours, and the sequence is then repeated. The average application rate is 2.8 mgd per field. Field "A" is a "recovery field;" runoff from the primary fields is trapped in a basin to prevent it from entering the Laguna de Santa Rosa. The runoff is then applied to field "A" by five self-propelled sprinkler tractors.

The Alpha and Kelly irrigation systems each have slightly different recovery systems, but the basic purpose is the same: to prevent irrigation runoff from leaving the property and entering the Laguna.

CONCLUSIONS

The three reclamation projects discussed above were all undertaken for different purposes and have all faced different problems in implementation. There are, however, some general conclusions which can be cited.

- Developing a market for reclaimed water requires a long, iterative effort which moves from general volume, quality, and price parameters to more specific values for these items. Initially, water prices may not be known and the volume for which there will be a demand can be only estimated. Further, while the chemical quality of the water can be obtained rather quickly, the quality requirements of the user may not be known. This information must be developed slowly and users contacted as each cycle is completed. Trying to go directly from preliminary planning to firm commitments will probably lead to implementation problems.
- Chemical quality of the water is often a problem from a technical, rather than public health, standpoint. Salts, phosphorus, boron, are chemical constituents which presented problems for each of the three projects discussed in this paper, and it is likely that additional treatment and or management techniques will be required for many water reclamation programs.
- Conveyance distance is usually the factor which limits the economic feasibility of irrigation reuse. The three projects discussed above were fortunate to have potential reuse sites located nearby. Many, probably most, wastewater treatment plants are located in areas remote from potential reuse sites.

Table 1. Mineral Quality of Effluent and Major Influent Streams of San Jose-Santa Clara WPCP

Constituent	Constituent concentration, mg/l unless otherwise noted ^a			
	San Jose/ Santa Clara ^b WPCP effluent ^c	Combined San Jose/ Santa Clara influent ^b	Santa Clara Northside Pumping Station influent ^b	Milpitas Sanitary District influent ^c
Major cations				
Ammonium, NH ₄	17	30	32	30
Calcium, Ca	61	52	89	33
Magnesium, Mg	34	33	31	29
Potassium, K	19	22	17	12
Sodium, Na	240	207	236	196
Major anions				
Bicarbonate, HCO ₃	323	342	332	295
Chloride, Cl	325	278	295	214
Nitrate, NO ₃	6.2	0.19	0.64	0.13
Phosphate, PO ₄	14	11	32	29
Sulfate, SO ₄	55	57	248	81
Other constituents				
Total dissolved solids (TDS)	988	896	1,060	748
Total alkalinity as CaCO ₃	265	280	272	242
Total hardness as CaCO ₃	152	266	222	201
Specific conductance, umho/cm	1,744	1,679	2,153	1,407
Boron, B	0.6	0.6	0.7	0.7
Fluoride, F	2.4	0.66	15	0.42
Silica as SiO ₂	27	15	28	16
pH, units	7.8	6.4	7.0	7.0
Sodium adsorption ratio (SAR), units	6.1	5.5	5.5	6.0
Adjusted SAR, units	13.7	12.5	12.8	12.5

^aTwenty-four hour composite samples collected on 7/13-13/77.

^bConstant volume composite.

^cFlow-proportional composite.

^dValue is probably not valid. Ionization interferences were encountered in this determination.

Table 2. Design Data for Milpitas-North San Jose Project

Item	Stage 1	Stage 2
Projected demands, mgd		
Average annual	6.9	12.4
Peak day	13.0	16.4
Peak hour	28.5	31.4
Interstage pumping		
Design flow, mgd	13.0	16.4
Pumps		
Number	2	2
Capacity, each, mgd	13.0	16.4
Total head, ft	30	30
Disinfection		
Design flow, mgd	13.0	16.4
Chlorine contact tank		
Number of tanks	1	1
Number of passes/tank	4	5
Length per pass, ft	120	120
Width per pass, ft	15	15
Average water depth, ft	15	15
Detention time, minutes	90	90
Chlorine dose, mg/l		
Assumed average	2	2
Maximum	6	6
Storage		
Ground-level storage basin		
Number	1	1
Storage volume, mg	7.0	7.9
Maximum water depth, ft	15	15
Elevated storage tank		
Number	1	1
Storage volume, mg	0.3	0.3
Maximum water surface elevation, ft above M.S.L.	205	205
Reclaimed water pumping		
Design flow, mgd	28.5	31.4
Pumps		
Number	6	6
Capacity, each, mgd	5.7	6.3
Total head, ft	300	300

Table 3. Potential Reuse Applications in Napa-American Canyon Study Area

Use category	Treatment level ^a	Comments
Vineyard		
Drip irrigation	S	• Additional on-line filtration, with frequent cleaning, may be required
Frost protection	S	• DHS must approve first Runoff must be contained
Spray irrigation	T	• For period when fruit is present on vines • Effluent from tertiary may not meet DHS definition of "Filtered Water": 2.0 turbidity units, average
Fodder, fiber, seed crops irrigation	S	• DHS regulations specify primary effluent
Pasture, milking cattle	S ^b	• Adequate drying must occur prior to allowing animals in property
Landscape irrigation	S ^b	• Minimize public exposure
Golf course	S ^b	• DHS may require tertiary (T) level
Freeway medians	S ^b	
Schools, parks, hospital grounds	T	• Proposed DHS regulations require that effluent be "essentially pathogen-free." It is anticipated that tertiary facility effluent will be adequate

^a S = Secondary, or pond, effluent from NSD and ACCWD oxidation ponds.
T = Tertiary effluent from new NSD-ACFWD joint treatment facility.

^b Modifications to existing disinfection facilities may be required to meet coliform limitation of 23 MPN/100 ml.

Table 4. Potential Reuse Demand in Napa-American Canyon Study Area

Irrigation subarea	Water demand, acre-ft/yr							Total
	Vineyards		Pasture	Silage corn	Orchards	Landscaping	Tanneries	
	Drip irrigation	Frost protection						
1990								
Cathlamet	82	912	3,120	0	150	0	0	3,664
East	0	0	2,080	0	0	0	0	2,080
Southeast	0	0	2,080	0	0	0	0	2,080
Northeast	0	0	0	0	0	458	295	753
Total	82	912	7,280	0	150	458	295	8,577
2000								
Cathlamet	231	936	2,080	1,250	260	121	0	4,880
East	0	0	1,907	1,146	0	0	0	3,053
Southeast	0	0	1,731	1,042	0	0	0	2,773
Northeast	0	0	0	0	0	1,340	390	1,430
Total	231	936	5,718	3,438	260	1,161	390	12,138

Table 5. Design Data, Recommended Napa-American Canyon Projects

Parameter	Value	Parameter	Value
Carneros Subarea, Alternative F3		Alternative F3 (continued)	
Flow		Equalizing reservoirs	
Annual average, acre-ft/yr	3,700	Number	1
Peak, mgd	16	Capacity, mil gal	1.0
Distribution system force main length, ft		Turnouts for customers	45
36-in. diameter	14,300	Southeast Subarea, Alternative V5	
30-in. diameter	24,100	Flow	
18-in. diameter	4,200	Annual average, acre-ft	1,040 ^b
14-in. diameter	3,000	Peak, mgd	4.7
12-in. diameter	14,500	Conveyance force main length, ft	16,600
Total	60,100	Diameter, in.	24
Storage reservoir volume, acre-ft	1,500 ^a	Pumping station	
NSD ponds pumping station		Number of pumps	3
Number of pumps	3	Total capacity, mgd	7.5
Total capacity, mgd	16	Total dynamic head, ft	100
Total dynamic head, ft	240	Storage reservoir volume, acre-ft	2,500
Huichica pumping station		Irrigated land, acres	
Number of pumps	3	Initial	300
Total capacity, mgd	16	Ultimate	1,200
Total dynamic head, ft	190		

^aTotal volume of Huichica Reservoir will be 5,000 acre-ft; 1,500 acre-ft will be reserved for Napa-American Canyon Project.

^bInitial flow; ultimate flow will be 4,160 acre-ft/yr.

Table 6. Effect of Napa-American Reuse Project on Wastewater Treatment Costs

Parameter	Alternative F3	Alternative V5	Tannery waste treatment	Total
Reuse project annual cost, thousand dollars/yr				
Total ^a	845	625	140	1,610
Local share ^b	215	200	50	375
Operation and maintenance only	125	35	35	195
Cost savings from reduced operation of tertiary facility, thousand dollars/yr ^c	<370>	<105>	<30>	<505>
Net cost of reuse alternative, thousand dollars/yr				
Total	475	520	110	1,105
Local share	<155>	95	20	<130>
Operation and maintenance only	<245>	<70>	5	<310>

^aAmortized capital costs plus operation and maintenance expenses; 1980 levels.

^b12.5 percent of total capital costs, plus operation and maintenance expenses.

^cBased on \$100/acre-ft (1980 levels).

Table 7. Chemical Characteristics of Pilot Irrigation Study Water Supplies

Parameter	Concentration in milligrams per liter		
	City water ^a	Well water ^a	Secondary effluent ^b
<u>Anions</u>			
Nitrate (NO ₃)	1.0	1.9	19
Chloride (Cl)	4.6	128	118
Sulfate (SO ₄)	14	189	67
Bicarbonate (HCO ₃)	150	198	163
Carbonate (CO ₃)	0	0	0
H-Phosphate (HPO ₄)	-	-	15
H ₂ Phosphate (H ₂ PO ₄)	-	-	3.3
<u>Cations</u>			
Ammonium (NH ₄)	-	-	13.2
Sodium (Na)	10.2	93	90
Potassium (K)	1.4	2.3	10
Calcium (Ca)	27	68	40
Magnesium (Mg)	16	40	19
Boron	0.2	0.1	0.53
TDS	170	695	468
Total Nitrogen (N)	-	-	19
Total Phosphate (PO ₄)	-	-	19
SAR	0.38	2.21	2.93
SAR (adjusted)	0.48	4.12	4.79

^a Average values for four samples collected over a two year period.

^b Average values for six 24-hour composite samples collected over a two year period.

Table 8. 1974 Crop Yields, Santa Rosa Pilot Study

Plot No.	Crop	Irrigation water	Yield, tons/acre		Yield ratio effluent/control	
			Fresh crop	Dry matter	Fresh crop	Dry matter
1	Pasture	Well	14.7	2.8		
2	Pasture	Well	5.9	1.7		
29	Pasture	Well	4.4	0.7		
		Mean	8.3	1.7		
3	Pasture	Effluent	7.0	1.2		
13	Pasture	Effluent	2.6	0.6		
17	Pasture	Effluent	5.9	1.1		
		Mean	5.2	1.0	.63	.59
5	Corn silage	City	19.7	4.1		
11	Corn silage	City	20.5	3.4		
22	Corn silage	City	21.8	3.9		
		Mean	20.7	3.8		
16	Corn silage	Effluent	17.4	3.6		
21	Corn silage	Effluent	20.7	4.0		
26	Corn silage	Effluent	21.8	4.3		
		Mean	20.0	4.0	.97	1.05
6	Barley silage	City	12.9	4.5		
7	Barley silage	City	9.8	2.8		
9	Barley silage	City	10.7	4.6		
		Mean	11.1	4.0		
12	Barley silage	Effluent	5.8	2.2		
20	Barley silage	Effluent	4.0	1.5		
25	Barley silage	Effluent	7.7	2.9		
		Mean	5.8	3.2	.52	.80
18	Sudan	City	18.0	4.0		
19	Sudan	City	19.6	4.3		
24	Sudan	City	19.6	4.3		
		Mean	19.1	4.2		
4	Sudan	Effluent	4.4	1.0		
27	Sudan	Effluent	8.9	2.0		
30	Sudan	Effluent	6.5	1.4		
		Mean	6.6	1.5	.34	.36
8	Corn silage	Effluent	19.8	4.1		
14	Corn silage	Effluent	19.2	3.9		
23	Corn silage	Effluent	18.4	4.2		
		Mean	19.2	4.1	.93	1.1
10	Barley grain	Effluent	1.5	1.3		
15	Barley grain	Effluent	1.5	1.3		
28	Barley grain	Effluent	0.7	0.6		
		Mean	1.2	1.1		
33	Sudan	Effluent	13.9	3.1	.73	.74

Table 9. 1975 Crop Yields, Santa Rosa Pilot Study

Crop	Irrigation water	Fertilizer application rate, lbs/acre		Yield tons/acre		Yield ratio effluent/control	
		N	P ₂ O ₅	Fresh matter	Dry matter	Fresh matter	Dry matter
Pasture	Well water	150	150 ^a	9.2	1.8	-	-
Pasture	Effluent	0	100	9.6	1.9	1.0	1.1
		50	100	10.4	2.1	1.1	1.2
		150	100	11.8	2.3	1.2	1.3
Corn silage	Effluent	0	65	21.3	3.4	1.3	1.3
		50	65	24.5	3.9	1.1	1.1
		100	65	24.2	3.9	-	-
		150	65	22.5	3.6	1.1	1.1
Corn silage	City water	0	65	17.0	2.7	-	-
		50	65	22.2	3.6	-	-
		150	65	21.3	3.4	-	-
Barley grain	Effluent	0	50	0.9	0.8	0.6	0.6
		50	50	1.7	1.5	1.2	1.2
		100	50	1.6	1.4	1.1	1.1
		150	50	1.5	1.4	1.1	1.1
Barley grain	City water	150	50	1.4	1.3	-	-
Sudan	Effluent	0	100	10.0	1.8	-	-
		50	100	10.5	1.9	-	-
		150	100	16.7	3.0	-	-

^aPhosphorus fertilizer was applied to this plot prior to the 1974 irrigation season.

Table 10. Design Data, Santa Rosa Effluent Irrigation System

Parameter	Value	Parameter	Value
Treatment plant capacity, mgd	5	Irrigation pumps (continued)	
West College Avenue	15	Kelly system	
Laguna		Number	3
Effluent pumps		Capacity, mgd	
Number	4	No.'s 1, 2	3.8
Head, ft		No. 3 (recovery)	1.4
No.'s A1, A2	20	Head, ft	
No.'s B1, B2	34	No.'s 1, 2	346
Capacity, mgd		No. 3 (recovery)	65
No.'s A1, A2	15	Irrigated land area owned by	
No.'s B1, B2	10	City, acres	
Storage pond capacity, acre-		Alpha system	
ft		Primary	266
Meadow Lane ponds	1,920	Recovery	57
Alpha ponds	96	Brown system	
Arlington pond	14	Primary	287
Brown pond	218	Recovery	79
Kelly ponds	94	Kelly system	
La Franconi pond	18	Primary	366
Poncia pond	20	Pipeline length, ft	
West College I ponds	206	16-in. diameter	9,400
West College II pond	63	24-in. diameter	13,500
West College III ponds	113	27-in. diameter	54,000
West College IV pond	49	30-in. diameter	14,200
Irrigation pumps		33-in. diameter	13,000
Alpha system		42-in. diameter	12,500
Number	4	48-in. diameter	10,700
Capacity, mgd		Irrigation piping (2-in.	
No.'s 1, 2	2.9	to 16-in. diameter)	690,000
No.'s 3, 4 (recovery)	1.5		
Head, ft			
No.'s 1, 2	267		
No.'s 3, 4 (recovery)	261		
Brown system			
Number	4		
Capacity, mgd			
No.'s 1, 2	3.2		
No.'s 3, 4 (recovery)	1.4		
Head, ft			
No.'s 1, 2	283		
No.'s 3, 4 (recovery)	150		

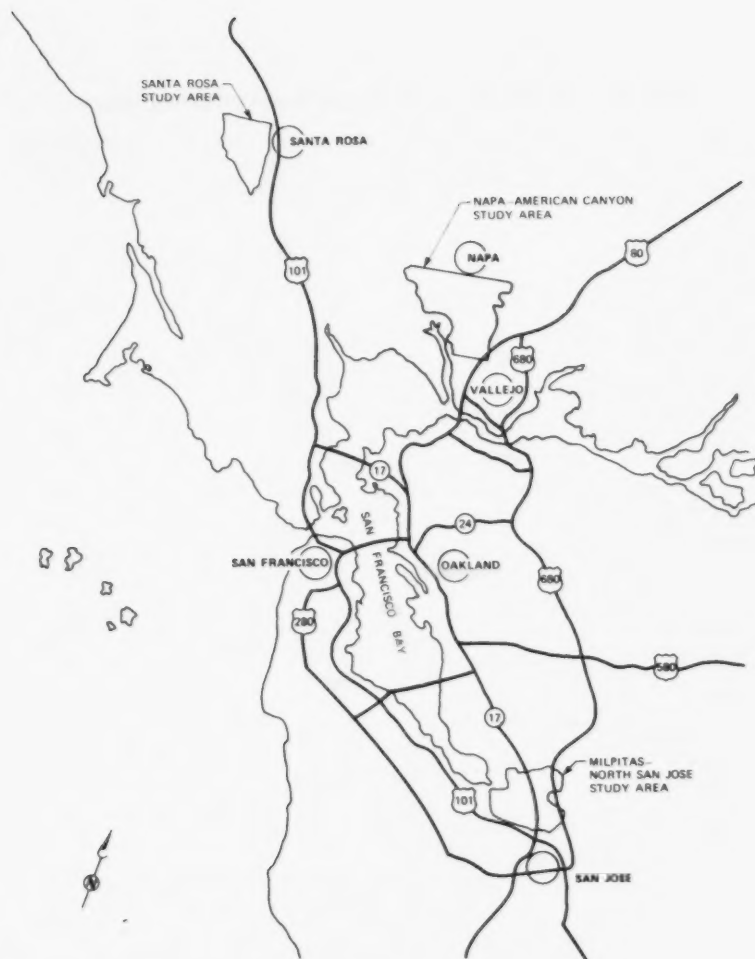


Fig. 1 Reclamation Project Study Areas

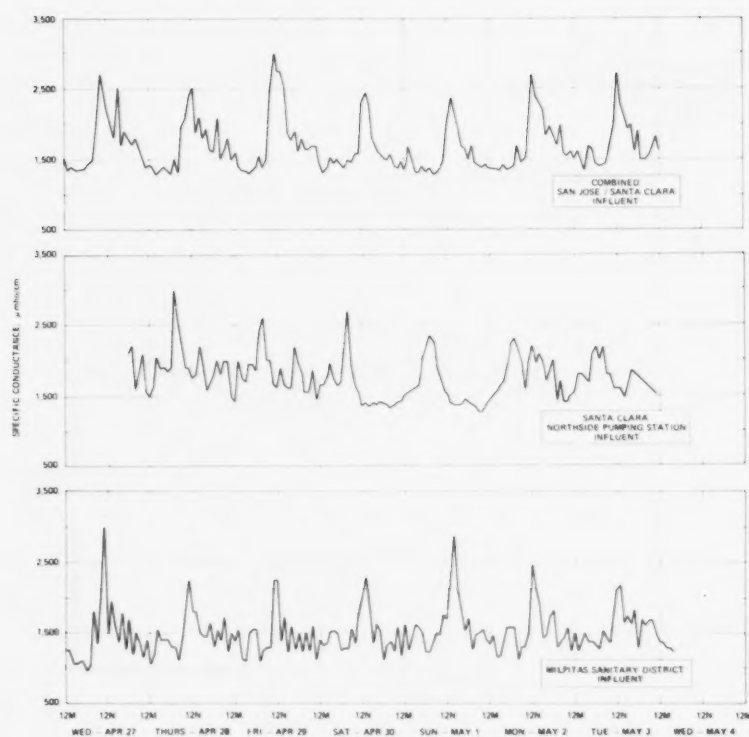


Fig. 2 Specific Conductance of San Jose-Santa Clara WPCP Influent Streams

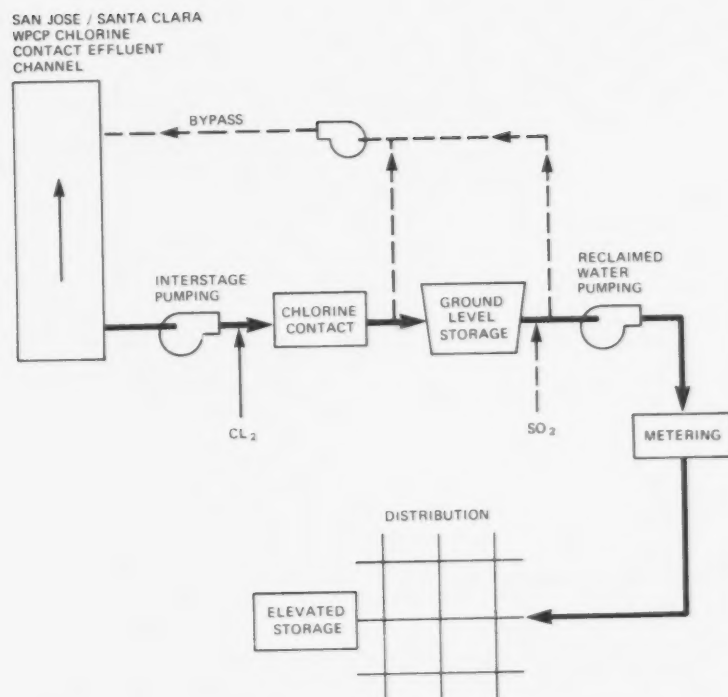


Fig. 3 Schematic Flow Diagram, Milpitas-North San Jose Reclamation Project

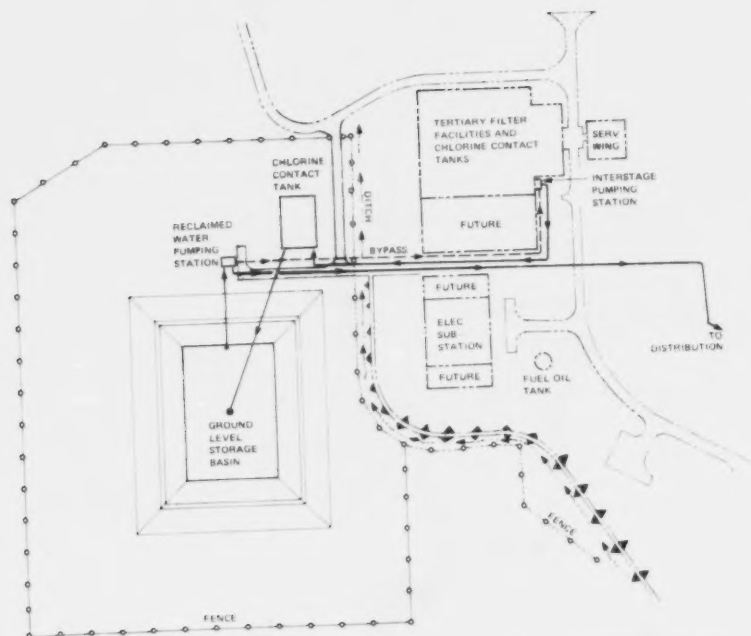


Fig. 4 Layout of Treatment, Storage, and Pumping Facilities for Milpitas-North San Jose Project

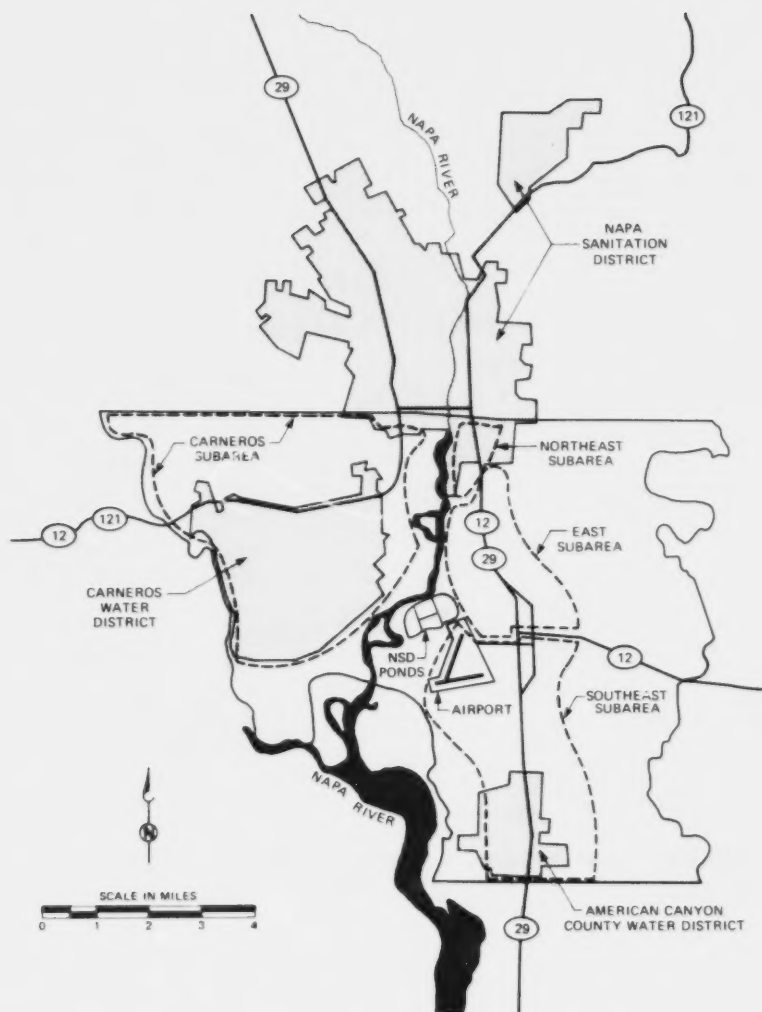


Fig. 5 Napa-American Canyon Study Area



Fig. 6 Dry Farming is Practiced in Much of the Study Area



Fig. 7 An Increasing Acreage is Being Converted to Vineyards

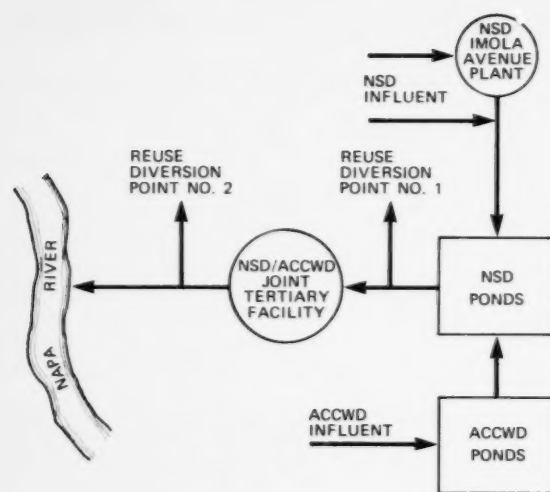


Fig. 8 Napa-American Canyon Treatment Facility Flow Diagram

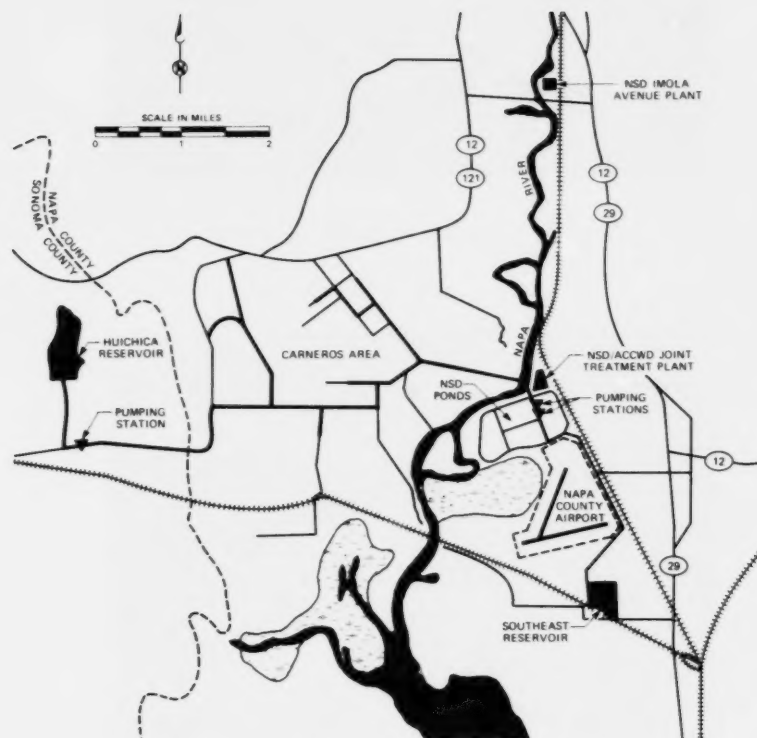


Fig. 9 Recommended Project, Napa-American Canyon Study

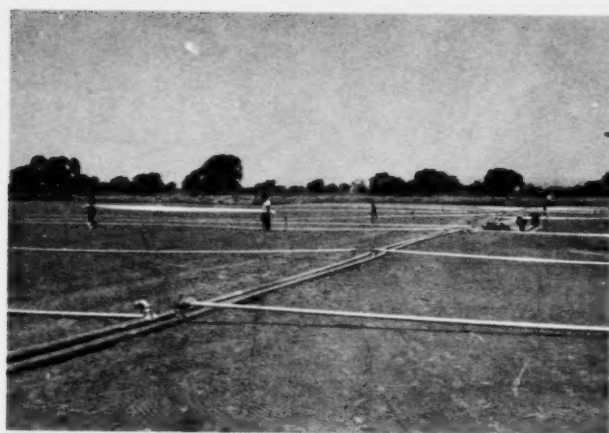


Fig. 10 Setting Up Irrigation Equipment



Fig. 11 Irrigation at Study Site

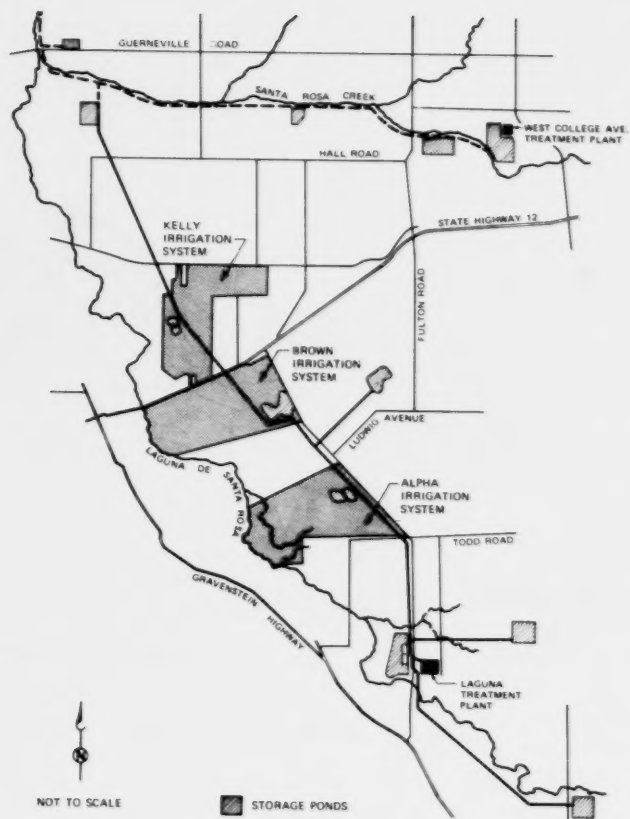


Fig. 12 Layout of Santa Rosa Irrigation System



Fig. 13 Brown Irrigation Fields

CLOSED LOOP RECYCLE AND REUSE OF WATER CLOSET FLUSH WATER

Clinton E. Parker
Associate Professor
University of Virginia
Department of Civil Engineering
Charlottesville, Virginia

John W. Reynolds
Perkin-Elmer Corp.
Houston, Texas

OBJECTIVE

The objective of this research and development work was: (1) establish a treatment train for water closet water recycle-reuse employing biological treatment by extended aeration followed by granular media filtration and (2) evaluate the adequacy of biological treatment for treating water recycled 10 to 20 times in a closed system, and the acceptability of the recycled water as a flush fluid at 95% reuse.

INTRODUCTION

From the analysis of the operation of extended aeration treatment systems, it was concluded that removal of suspended solids from extended aeration effluent would produce a water of a quality suitable for flushing toilets. It was projected that 90% to 95% of the water used at toilet facilities goes to flush water closets and that 10 to 20 recycles would be required to reach an equilibrium with sewerable potable water from hand basins and other miscellaneous uses (1).

Since no information was available to support satisfactory operation of a biological treatment system where water was recycled through the system 10 to 20 times, the research and development of a recycle-reuse concept was evaluated by a bench-scale model that used a synthetic wastewater comparable in characteristics to waste inputs from public toilet facilities (2).

Over a five-year period, bench-scale biological-physical treatment trains were evaluated to determine the suitability of this technology in the closed-loop treatment of water closet wastewater where the flush water is continuously recycled and reused an average of 20 times. Results from the laboratory operation of a completely mixed suspended biological growth unit followed by sand filtration indicated that biological oxidation would not be impaired and flush water reuse at the 95% recycle level would be clear, odor free, and acceptable to the user. Based on laboratory results a full-scale prototype was installed to demonstrate field reliability of the concept, to further delineate design criteria, and to test public acceptance. In November 1976 construction of a recycle addition to an

existing extended aeration biological treatment unit was completed and put into operation on March 15, 1977, and evaluated over a period of 18 months (3).

RECYCLE-REUSE SYSTEM

The system used in this research was biological treatment by extended aeration followed by rapid filtration using a granular nonhydrous aluminum silicate media. Wastewater from the toilet facility was treated by the 37.8 m³/d extended aeration system and released to a small stream prior to the installation of the filtration system. Modification for water recycle-reuse involved pumping the biologically treated effluent through a newly constructed filter and then back to the water closets as the flush fluid. The filter was 1.83 meters in diameter and was operated at a filtration rate of 94 lpm/m². Filter cleaning rates were 1221 lpm/m² for surface wash and 346 lpm/m² for backwash. Water closet flush water was passed through biological treatment and rapid filtration in a closed loop system. Potable water needs were served by a well through a different piping system. Figure 1 is a schematic of the system used at the toilet facility. Potable water used for wash basins and other uses were added to the closed loop system and an equivalent amount was released to an evaporation pond.

RESULTS

The field evaluation of the recycle-reuse concept was divided into two study phases: (1) start-up phase and (2) equilibrium phase. Water passed through the water closets for each phase is presented in Table 1.

The start-up phase was between March 15, 1977 and August 31, 1977. Operational data on the biological system during this phase is presented in Table 2. Characteristics of the recycled flush water used in the water closets is shown in Figures 2, 3, 4, and 5.

The equilibrium phase evaluation was made between September 1, 1977 and August 31, 1978. Table 3 shows the results for the biological treatment unit for this period. The variation in the flush water quality just before reuse is presented in Figures 6, 7, 8, and 9. Percent recycle experienced during equilibrium is given in Table 4.

DISCUSSION OF RESULTS

Biological Treatment Performance

The biological treatment system proved to be durable and effective in biological oxidation of biodegradable organics, thus providing an odor-free and chemically stable water. Nitrogen concentrations were high, however, they did not prove to be toxic to the biomass. Most pronounced changes in the biological unit were responses to pH, alkalinity, temperature and water input.

Oxygen uptake by the biomass responded to expected changes in temperature and utilization by nitrifiers. The range of values experienced during start-up and equilibrium were approximately the same with the exception of July 1978 and August 1978. In July a substantial transient increase of alkalinity in the raw wastewater was experienced. With the high alkalinity and the accompanying increase in pH significant nitrification occurred. The result was an increase in oxygen uptake and a lowering of dissolved oxygen in the aeration basin, a significant lowering of ammonia nitrogen and nitrite nitrogen, and an increase in nitrate nitrogen. Although an increase in nitrification was observed during start-up in late summer, it was not to the extent experienced in July and August during the equilibrium phase. Since the trend for the latter part of August was a decrease in nitrification, the flourish of nitrification appeared to be solely a response to increases in pH and alkalinity.

The lowering of the MLSS during the equilibrium phase was a response to sludge wastage and the use of make-up water in an attempt to maintain 95% recycle. The most significant change coincided with the addition of make-up water and the increase in nitrification in July. Although low MLSS values were experienced (and effective) during late summer, the system operated with a MLSS between 3,000 and 5,000 mg/l and settleability (30 minute settling in a litre graduated cylinder) of 200 to 800 ml during most of the study.

The biomass under aeration was mostly filamentous but separation of the MLSS from the liquid fraction was good and suspended solids carry over from the settling basin was not sufficient to be detrimental to the granular filter performance. Physical appearance of the MLSS and the ability to separate the solids from the liquid phase indicated that average water reuse of about 20 times was acceptable and sustained reuse of up to 30 times may cause deterioration in suspended solids separation and ultimately recycle flush water quality.

Alkalinity and pH in the biological unit followed the same pattern established in the start-up phase. Extreme alkalinity changes were the result of the inputs from the raw wastewater. Variation in alkalinities observed were extreme, 17 to 563 mg/l, however, the operating range was almost solely between 20 and 100 mg/l. The values of pH ranged from 5.1 to 7.9. High pH values were transient and most often the system maintained a pH between 5.5 and 6.5. Flush water quality remained acceptable through all the observed alkalinity and pH variations.

Data from the biological unit indicate that a biological system (extended aeration) used to treat recycled water closet flush water at 95% recycle can be routinely monitored by settleable solids, MLSS, pH, alkalinity, temperature and dissolved oxygen, and operation does not differ significantly from the operation of a system treating non-recycled water. Nitrogen and biological

oxygen demand will be the two parameters that will exhibit the greatest difference. Nitrogen will remain in the system primarily as ammonia nitrogen and nitrite nitrogen. Only when high alkalinity wastes are sewerred (enter the recycle loop) will significant increases in nitrate nitrogen be detected. Concentrations of ammonia nitrogen, nitrite nitrogen, and nitrate nitrogen in the aeration basin as high as 150, 240, and 125 mg/l, respectively, were not detrimental to biological degradation. If significant agitation of the flushed raw wastewater between the flush and the treatment system is provided, an equilibrium concentration of ammonia nitrogen can be expected except when transient high alkalinities occur. (Introduction of waste by the water closet user raises the pH and results in ammonia stripping when some form of agitation is applied.) At this site, flow in the sewer between the water closet facility and the biological unit provided sufficient agitation for maintaining non-toxic equilibrium nitrogen concentrations. As a result of the high ammonia nitrogen and nitrite nitrogen maintained in the system, BOD values were of no real significance as a measure of biodegradable organics.

For aesthetic reasons and to warn the user that the water was recycled, blue food coloring (FD & C No. 1) was added to the system. The dye was not degraded, hence, addition was only periodically required and then in small amounts.

Foaming during this study was insignificant and, as a result, no chemical additives were required for control.

Filter Performance

Suspended solids concentrations imposed upon the filter were variable, however, they were usually maintained between 60 and 20 mg/l. Suspended solids in the flush water were generally less than 20 mg/l. Only during the period of freezing and extremely cold weather was a high suspended solids value, 41 mg/l, detected in the flush water.

Filter performance was significantly affected by backwash frequency. The filter system was operated in a pressure differential (across the filter bed) backwash mode until May 1978. Due to filter over design it could operate in this mode without backwash for sustained periods, 4 weeks or more, unless backwashed manually. (Manual backwash while in the pressure differential mode was occasionally performed as a check on the system.) After setting the system on an automatic weekly backwash mode, except for one analysis, flush water suspended solids did not exceed 12 mg/l.

Quality of Recycled Flush Water

During this study the appearance and chemical stability of the water remained acceptable as a flush fluid; no odors were detected and facility users readily accepted

the recycled water.

Only minor quality differences were detected between the start-up phase and the equilibrium study phase. Suspended solid (SS) differences between the two study phases were insignificant. Average suspended solids were approximately 15 mg/l for both study phases with the lowest suspended solids occurring in this work when the filter was backwashed a minimum of once weekly. (Suspended solids were almost consistently less than 12 mg/l during this period.) Total solids (TS), total dissolved solids (TDS), and total fixed solids (TFS) showed little differences between equilibrium values established in the equilibrium phase and start-up, except where ice and snow melt, entered the system and the adjustment in April 1978 to maintain 95% recycle. These parameters dropped to between 1,700 and 2,000 mg/l when water addition was sufficient to establish recycle ratios of 95% or less. In August when the recycle level rose to 96.3%, the solids returned to previous values, TS of approximately 2,400 mg/l. Chlorides rose slightly during the period where 96.7% recycle was maintained, however, again like solids data (with ice and snow melt followed by maintaining a recycle of 95% or less), they were between 550 and 600 mg/l. Conductivity patterns basically followed the same trends as the TFS and chlorides. Changes in chemical oxygen demand (COD) and total volatile solids (TVS) followed similar trends. Values for these two parameters during the equilibrium phase and start-up were within approximately the same range until make-up water was added to maintain 95% recycle or less. Values for COD and TVS were lowered from the 300 to 400 mg/l range during the period where adjustment of the recycle was implemented.

As previously stated, results for alkalinity and pH and nitrogen concentrations were very much related. (Interdependence of these parameters for microbial aquatic systems is well recognized and is a consideration in biological treatment system design.) In the start-up phase, equilibrium alkalinity and pH for the flush fluid was higher than for the biological unit. This was also true for the equilibrium phase. Alkalinity and pH values in both phases were within the same limits (100 to 150 mg/l and 6.5 to 7.5, respectively) except when transient high alkalinities occurred in the flushed wastes, causing a shift in flush water alkalinity to the 200 to 300 mg/l range. Although some difference in nitrogen concentrations between the two study phases was observed, as would be expected, the greatest difference was observed when alkalinity and pH values increased. During start-up evaluation equilibrium values of approximately 160 mg/l for total Kjeldahl nitrogen (TKN) and ammonia nitrogen ($\text{NH}_3\text{-N}$), 180 to 220 mg/l for nitrite nitrogen ($\text{NO}_2\text{-N}$) and 75 mg/l for nitrate nitrogen ($\text{NO}_3\text{-N}$) were indicated. Prior to the nitrification experienced in July and August 1978, the range for these values were 100 to 150 mg/l, 70 to 140 mg/l, 100 to 180 mg/l, and 25 to 60 mg/l for TKN, $\text{NH}_3\text{-N}$, $\text{NO}_2\text{-N}$, and $\text{NO}_3\text{-N}$, respectively. Due to the range of conditions (seasonal use, water input balance, temperature,

pH, alkalinity and other waste characteristics) the system was exposed to during the equilibrium phase variations in the nitrogen values experienced represent likely operative characteristics. In addition the nitrification response to increase alkalinity and pH during July and August, (where TKN and $\text{NH}_3\text{-N}$ was lowered to about 25 mg/l, $\text{NO}_2\text{-N}$ decreased to less than 5 mg/l and $\text{NO}_3\text{-N}$ increased to the 70 to 110 mg/l range) is also a likely operating response. The lowering of the $\text{NH}_3\text{-N}$ during late summer also appears to be related to an increase in $\text{NH}_3\text{-N}$ losses in the sewer system as a result of increase in temperature and flushed waste pH. (It should be noted that if $\text{NH}_3\text{-N}$ exhibited an inhibitory affect on nitrification, increase losses of $\text{NH}_3\text{-N}$ could result in increase nitrification. This is not an unlikely condition since total nitrogen decreased during the nitrification experienced in July and August.)

Flush water phosphorus content showed an increase between the start-up phase and the equilibrium phase, a rise from 20 to 40 mg/l to 40 to 50 mg/l with an increase in July and August of 60 to 70 mg/l. During the equilibrium phase raw phosphorus was generally between 30 and 60 mg/l with an occasional value much greater, however, in the equilibrium phase raw phosphorus was usually 30 mg/l or less with the exception of a rise to between 50 and 60 mg/l for late summer. A rising trend of raw wastewater phosphorus with a resulting rise in flush water occurred, however, variations in phosphorus concentrations to the extent exhibited were unaccountable. Interference with the phosphorus analysis and/or variation in phosphorus in the MLSS (influenced by sludge wastage, MLSS concentration and chemical characteristics of the recycle water) are plausible explanations.

SUMMARY

The plumbing at an existing public toilet facility using an extended aeration wastewater treatment system designed to treat $37.8 \text{ m}^3/\text{d}$ was modified to separate potable water use from water closet flush water use. A granular pressure filter with pre- and postfiltration storage tanks and a hydro-pneumatic tank were installed in a closed loop to filter effluent from the extended aeration unit and to return it to the water closets as a flush fluid. Filter backwash water was collected in a separate storage tank and returned to the influent of the extended aeration unit during off-peak use of the facility. A blue food dye was added to the recycled water to assure public awareness and aesthetic conditions. A water balance equilibrium was attained at approximately 95% recycle.

Operation of the closed loop extended aeration and sand filter system for flush water recycle and reuse was similar to the conventional operation of these processes. The influence of nitrogen accounted for the only significant operating difference. Ammonia nitrogen transformation to nitrite and nitrate nitrogen resulted in a low operating pH in the biological unit (5.5 to 6.5) and

incomplete nitrification. Nitrogen buildup in the form of ammonia, nitrite, and nitrate was experienced but the equilibrium concentrations did not impair organic biological oxidation efficiency. The biological system functioned satisfactorily with fungi as the predominate biomass. Biological solids were threadlike; however, they readily separated through gravity settling. Wastage of biological solids in the treatment of recycled water did not significantly increase over wastage experienced with non-recycled treatment of water closet wastewater.

During start-up and at equilibrium the physical, chemical and biological quality of the water was acceptable for its intended use. The recycled water did not have any detectable characteristic odor or objectionable odor. At 95% recycle the potable water sewer from nontilet uses provided a daily nonrecycled quantity to the recycle system that assured a water quality equilibrium condition that was not detrimental to the biological treatment system.

Water wastage from the recycle system occurred by evaporation from the extended aeration units and the surface of holding tanks and through an overflow to a terminal holding pond. During the period of March 15, 1977, to August 31, 1978, $1.01 \times 10^6 \text{ m}^3$ of water were recycled and reused in the closed loop system.

CONCLUSIONS

Extended aeration biological treatment employed to treat recycled and reused water will function at conditions not normally experienced when these systems are used to treat domestic wastewater. The biological transformation of ammonia nitrogen to nitrate nitrogen will not be complete. Ammonia, nitrite, and nitrate nitrogen will accumulate in the system, however, since upon use flush water pH increases to about 8.3 and agitation can be used to enhance ammonia nitrogen loss, equilibrium nitrogen levels will not rise to a level that will be toxic to the heterotrophic organisms that can satisfactorily oxidize the organics. The system will satisfactorily operate at a pH of 5.5 to 6.5.

Wastage of biological solids in the treatment of recycled water does not significantly increase over the wastage experienced with non-recycled treatment of toilet facility wastewater. The current practice of using MLSS settleability as an operational test to determine the need for the wastage of biological solids can be continued.

The use of BOD test to determine biodegradable organics in a recycle-reuse system that employs biological extended aeration will not provide meaningful results. Five-day BOD values can be expected to be high and erratic.

This research conclusively demonstrated on a full-scale basis the acceptability of using biological oxidation and sand filtration as a treatment train for the

reuse of water closet wastewater with a recycle ratio of 20. For toilet facilities where stringent environmental controls prevent release of wastewater to a water course or where only a limited water supply is available, water recycle-reuse at the 95% level provides an economical alternative to the use of a mineral oil system.

Acknowledgements

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2. PARKER, C. E., RITZ, M. A., HEITMAN, R. H. & KITCHEN, J. D. Evaluation of a Water Reuse Concept for Highway Rest Areas. Transportation Research Record, TRB, National Academy of Sciences, Vol. 631 (1977) p. 37.
3. PARKER, C. E. Water Reuse at Highway Rest Areas: Evaluation Phase. Virginia Highway Research Council, VHTRC 78-R22 (December 1977).

TABLE 1

Flush Water Passed Through System

<u>Start-Up</u>		<u>Equilibrium</u>	
March 15 - August 31, 1977		September 1 - August 31, 1978	
<u>Gallons</u>	<u>Cubic Meters</u>	<u>Gallons</u>	<u>Cubic Meters</u>
882,900	3,337	1,791,000	6,770

TABLE 2

Start-Up Phase: Aeration Basin Analyses

Date 1977	Oxygen Uptake, mg/l/hr	MLSS, mg/l	MLVSS, mg/l	Settling, (a) ml	SVI (b)	Temp., Deg. C	DO, mg/l	pH
3/17	-	2300	1860	125	54	10	8.0	---
3/23	9.0	2900	2350	160	55	7	---	6.5
3/30	4.8	2650	2260	160	60	13	10.0	5.9
4/1	7.2	3040	2600	200	66	12	---	5.6
4/11	7.8	2630	2270	220	84	15	7.4	5.9
4/20	8.4	3310	2960	270	82	17	---	5.9
4/27	3.0	3040	2680	470	155	13	9.4	5.8
5/4	6.0	3480	3176	460	132	17	---	5.8
5/11	4.2	3720	3280	650	174	14	---	5.7
5/19	7.8	3990	3650	580	145	19	6.0	5.9
5/25	-	5150	4760	680	132	21	6.3	6.1
6/1	10.8	4750	4380	650	136	16	8.3	5.5
6/8	5.7	5540	5090	850	153	---	8.6	5.9
6/15	10.7	5130	4800	720	140	---	---	6.5
6/21	10.8	4100	3720	500	122	21	---	6.0
6/28	11.4	4780	4460	600	126	22	---	6.0
7/6	17.0	3720	3020	600	221	25	3.4	5.9
7/13	16.1	5540	5200	740	134	25	4.0	6.0
7/20	14.7	5590	5160	760	136	26	---	5.9
7/27	10.2	5440	5080	820	151	22	---	5.9
8/3	16.2	5650	5260	800	142	24	---	5.9
8/10	15.0	7040	6570	920	131	26	---	6.3
8/17	8.1	3990	3710	500	125	24	---	6.1
8/24	10.5	6457	6038	790	122	23	---	---
8/31	12.0	4372	4100	800	183	25	---	---

(a) Sludge volume occupied in a litre graduated cylinder after 30 minutes settling.

(b) Sludge Volume Index.

TABLE 3
Equilibrium Phase: Aeration Basin Analysis

Date	Oxygen Uptake, mg/l/hr	MLSS, mg/l	MLSS, mg/l	Settling, (a) ml	SVI (b)	Temp., Deg. C	DO, mg/l	pH	Alkalinity (as CaCO ₃) mg/l
1977									
9-07	10.2	9,268	9,052	750	176	25	9.3	5.9	71
9-14	7.6	9,112	9,512	850	166	23	5.2	5.7	58
9-20	7.7	9,312	9,090	700	162	19	6.5	5.6	92
10-13	6.6	9,756	9,520	800	213	12	6.8	5.3	11
11-02	6.9	9,712	9,090	780	185	15	6.0	5.2	21
11-16	6.0	9,576	2,800	580	225	17	9.2	5.9	10
11-10	6.0	---	---	720	---	8	9.8	5.6	38
12-07	5.3	9,368	3,236	650	193	7	9.8	5.5	33
12-13	---	3,416	3,280	650	---	---	---	6.6	96
1978									
1-04	5.2	3,576	3,396	780	218	9	---	7.3	267
1-25	9.6	7,796	2,660	600	215	2	---	7.2	192
2-08	---	3,728	3,658	500	139	6	---	7.9	592
4-15	6.0	9,808	9,658	---	---	6	8.5	6.9	268
6-29	9.6	9,956	9,658	800	161	10	9.8	7.2	256
9-12	---	3,928	3,680	650	167	15	8.5	6.9	171
9-26	8.0	2,576	2,947	200	78	9	8.5	6.3	70
10-10	9.7	3,320	2,180	200	66	19	7.0	6.6	96
10-25	9.2	3,300	2,850	300	100	19	5.0	6.3	67
11-11	8.6	3,096	3,329	350	100	20	5.1	6.0	58
1-07	7.6	3,336	3,036	320	100	19	5.3	6.0	59
3-18	10.8	3,527	3,512	530	169	18	2.9	6.1	56
5-24	19.0	2,896	2,868	380	127	22	9.7	6.1	59
7-28	17.2	3,180	3,096	390	123	23	5.0	5.9	46
9-15	26.8	3,058	2,907	360	117	21	1.8	7.0	359
11-19	26.9	3,120	3,816	220	109	21	0.9	7.3	563
1-19	26.8	2,609	2,009	250	109	22	1.2	6.3	79
3-26	20.0	2,677	2,205	280	113	23	0.8	5.7	25
5-02	19.8	2,768	2,428	400	100	23	1.1	5.5	17
6-09	26.5	2,652	2,132	290	123	23	0.7	5.7	37
8-16	27.0	2,852	2,026	280	115	26	2.3	5.6	25
9-30	15.6	---	---	280	---	26	3.1	---	---

(a) Sludge volume occupied in a litre graduated cylinder after 30 minutes settling.

(b) Sludge Volume Index.

TABLE 4

Equilibrium Phase: Monthly Recycle-Reuse

<u>Date</u>	<u>Recycle-Reuse Based on Actual Potable Water Entering Recycle System</u>
September 1977	96.7
October 1977 - March 1978	96.7
May 1978	94.2
June 1978	92.1
July 1978	95.0
August 1978	96.3

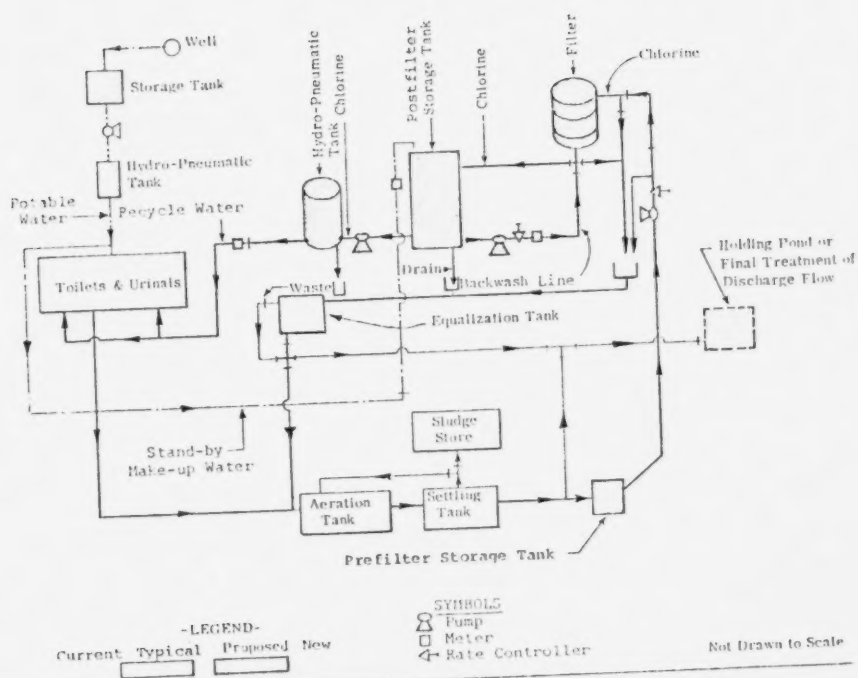


Figure 1. Schematic flow diagram for recycle and reuse of water to flush toilets

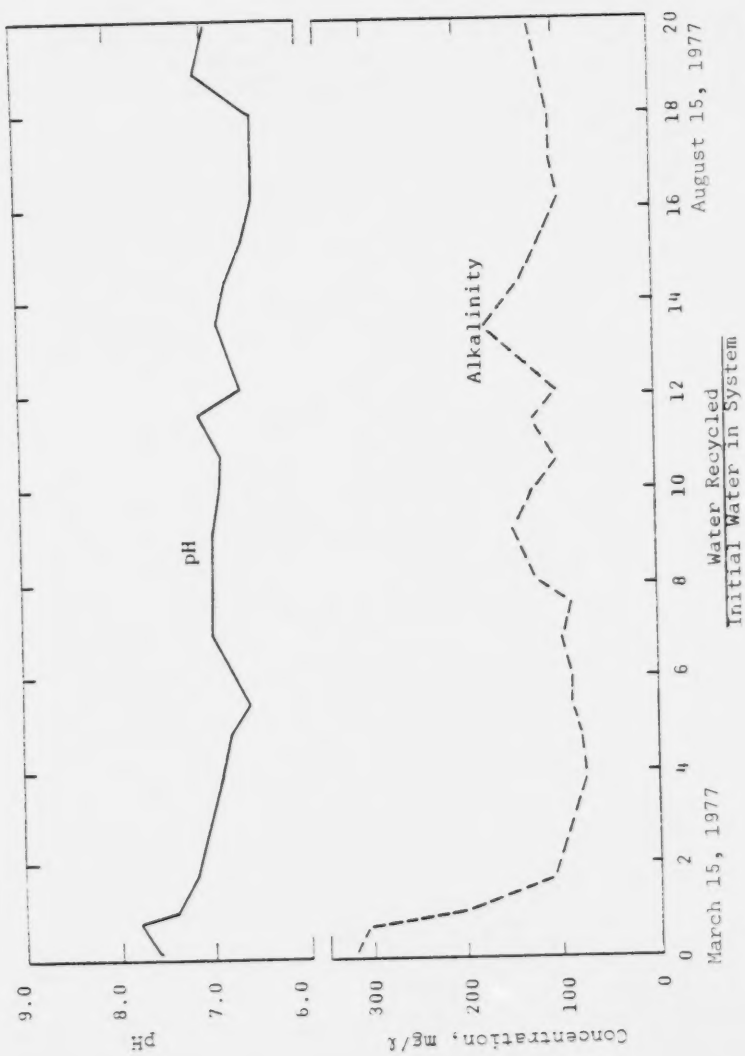


Figure 2. Start-Up Phase: flush water pH and alkalinity.

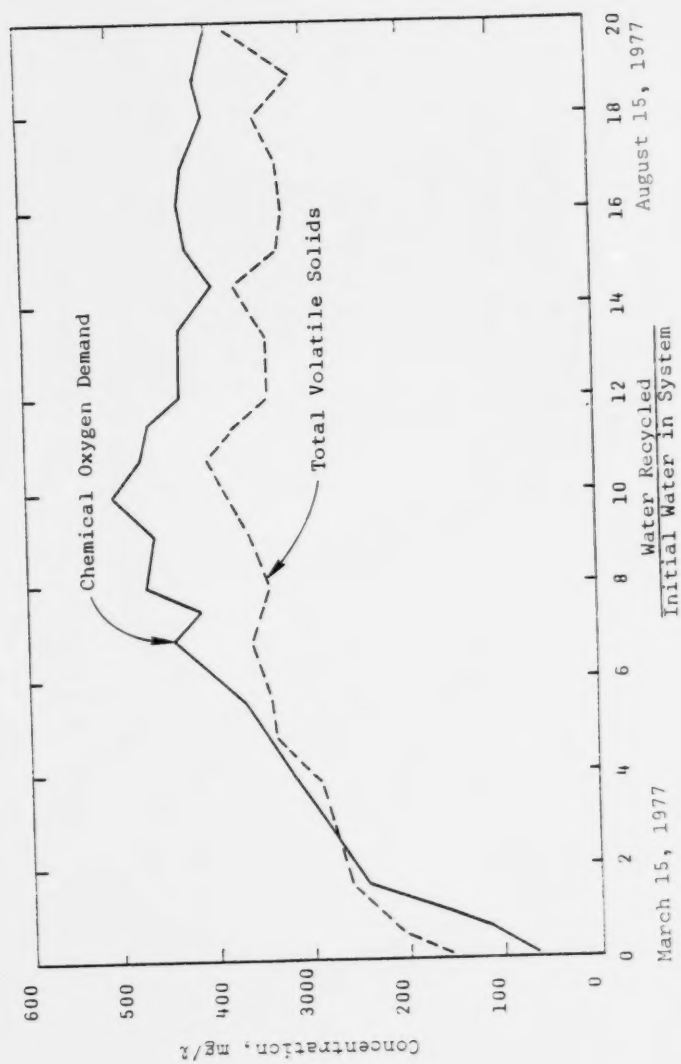


Figure 3. Start-Up Phase: flush water COD and volatile solids.

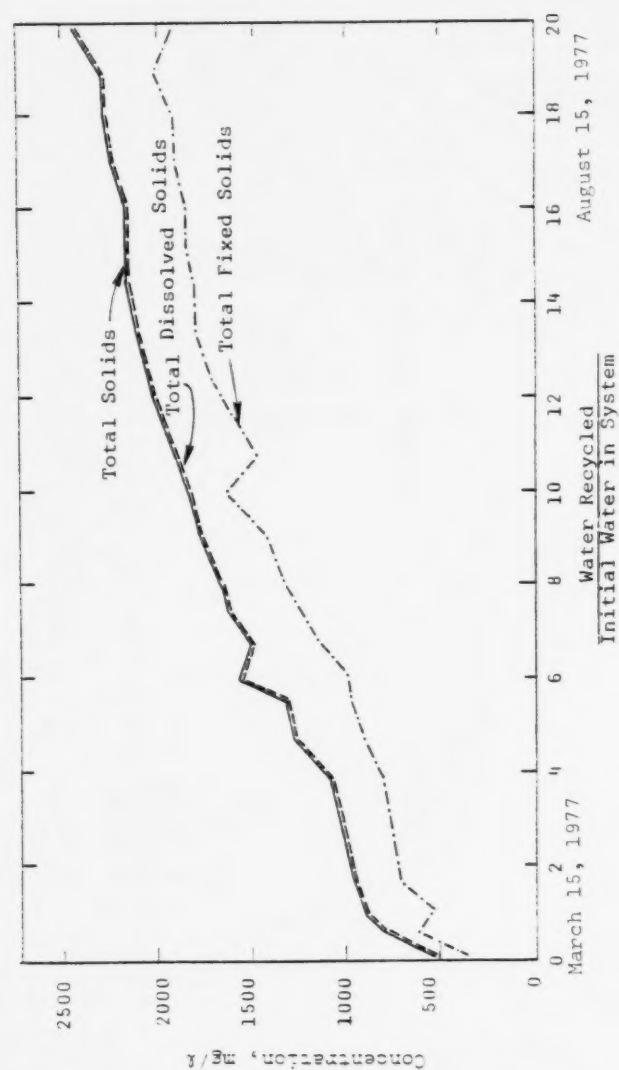


Figure 4. Start-Up Phase: flush water total solids.

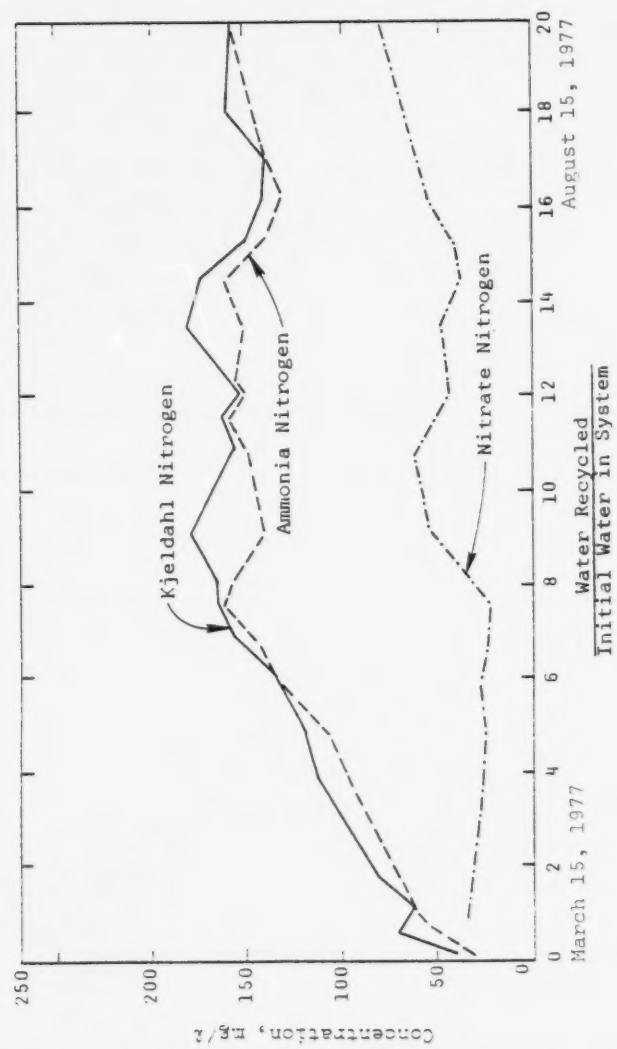


Figure 5. Start-Up Phase: flush water nitrogen.

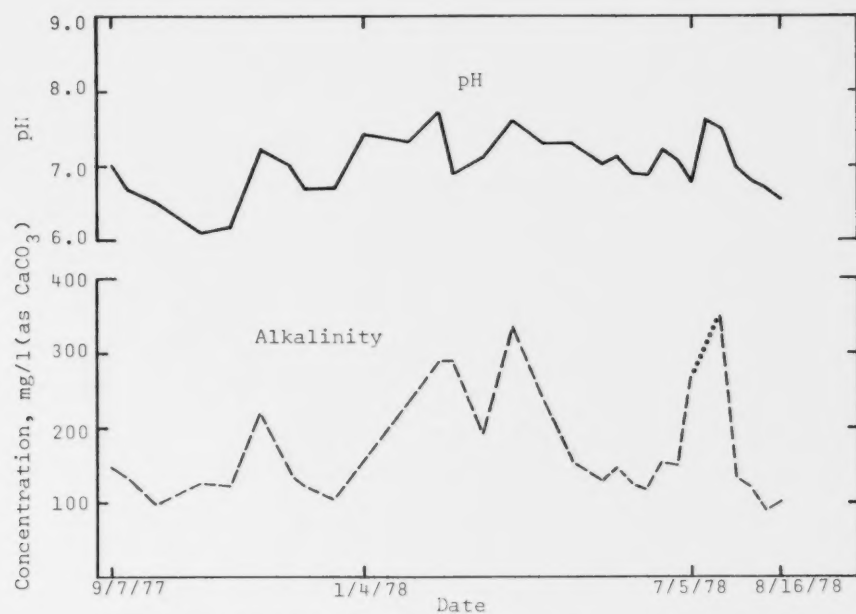


Figure 6. Equilibrium Phase: Flush Water pH and Alkalinity

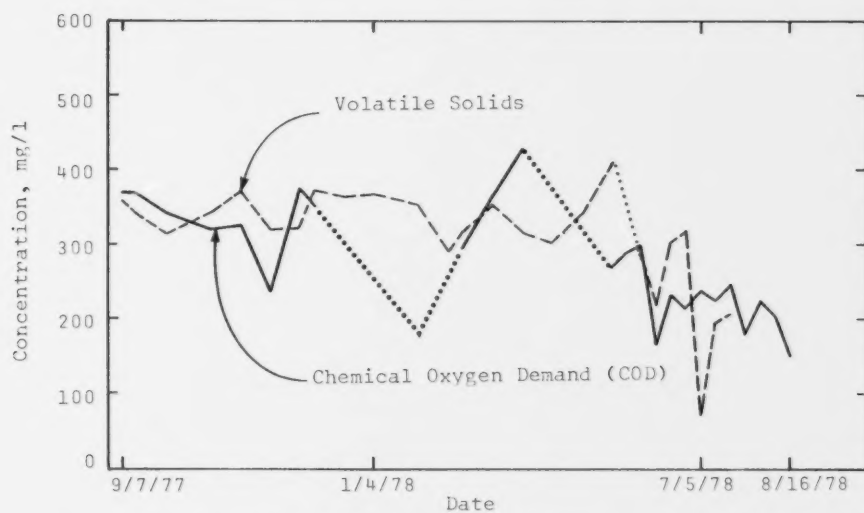


Figure 7. Equilibrium Phase: Flush Water COD and Volatile Solids

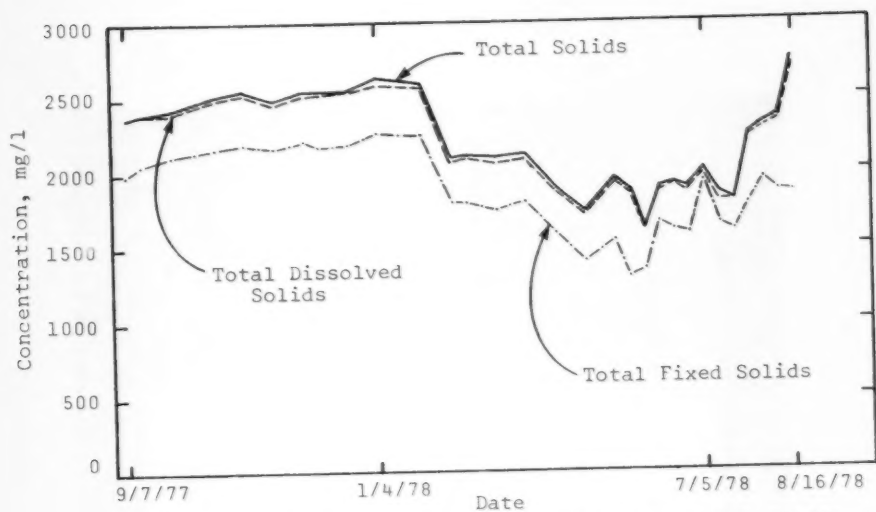


Figure 8. Equilibrium Phase: Flush Water Total Solids

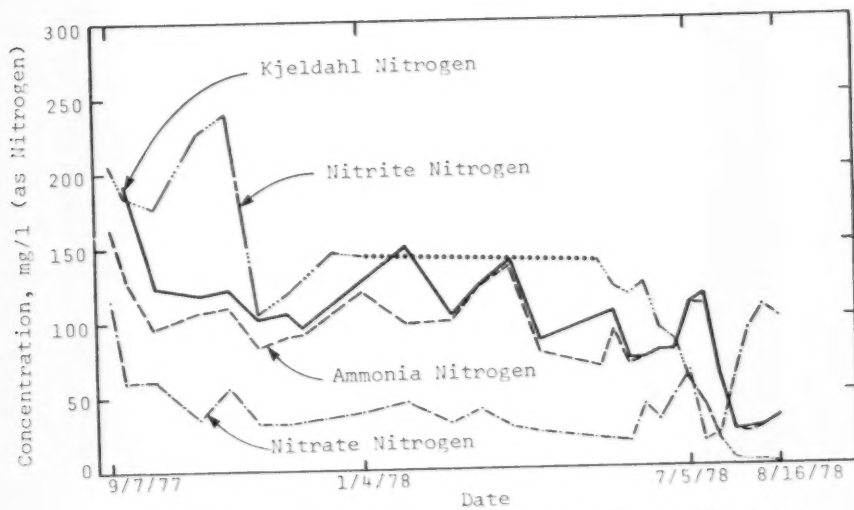
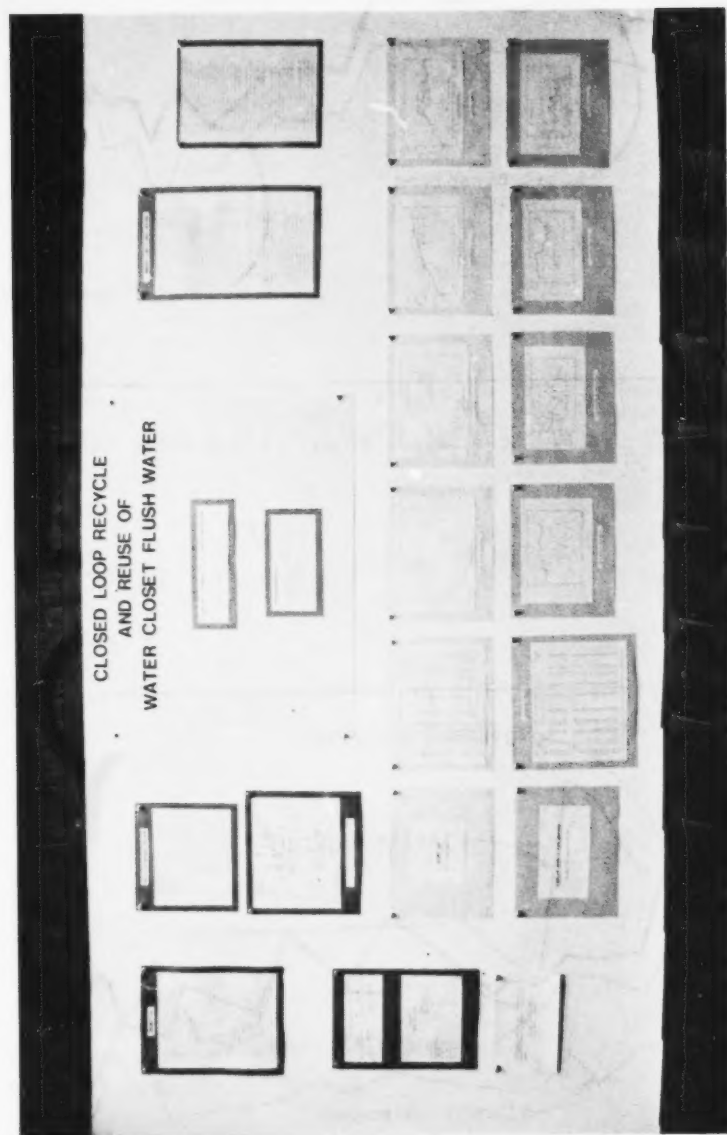


Figure 9. Equilibrium Phase: Flush Water Nitrogen



RECREATIONAL AREA WATER REUSE SYSTEM

Roy A. Ackerman
Technical Director
ASTRE
P.O. Box 5072
Charlottesville, VA 22905

I. A. Cosentino
President
Enviropak Division
Saxton Air Systems
129 Terwood Lane
Willow Grove, PA 19090

A zero discharge sanitary system, incorporating physical, chemical and biological processes, has been developed and tested. This system is capable of serving as a totally self-enclosed water recycle system, an advanced wastewater treatment facility, or as a part of an environmentally sound land application system.

The system, which is called the Enviropak AWTP System, is comprised of the following unit processes: equalization, gross particulate removal, a trickling filter, screening, mixed media filtration, activated carbon adsorption, disinfection, and storage. The AWTP installation at Olney Park (Maryland) has been operating for over a year. Four similar systems were simultaneously installed in Bucks County (Pennsylvania). In addition, the U.S. Army Corps of Engineers has chosen the land application version of the AWTP for the Blue Marsh Lake recreational facilities being constructed in Reading, Pennsylvania.

PROCESS DESCRIPTION

The basic system design, which can be conveniently scaled up or down, accommodates 250 flushes per day (plus the attendant gray water). Flush water in conjunction with wastewater from hand and face washing (and possible shower wastewater), enters the aerated equalization basin. This is designed to store loadings in excess of the design quantity for any given two day period. Aeration at 20 to 30 l/m³ min (20 to 30 cfm/1000 cu ft) is employed to effect mixing, as well as to prevent the formation of septic wastewater conditions.

From the equalization basin, the water is pumped to the gross-particulate removal unit. In the Maryland and Pennsylvania systems, this device is a vibrating shaker. The Blue Marsh Lake facility will utilize a sedimentation basin. The choice between the vibrating shaker and a sedimentation basin is determined by the flow volumes. (The Blue Marsh facility is designed for 65 m³/day (11250 gpd))

The sedimentation basin, if present, is sized to handle overflow rates of 20 m³/m²-day (480 gpd/sq ft). This provides a 3.7 hr retention time. These values, which are extremely conservative, provide more than sufficient capacity to effect a reduction in suspended solids.

Settled material is pumped from the basin floor on a timed basis, and transferred to the sludge storage tank.

The vibrating screen has two stages. As seen in figure 1, the wastewater from the equalization basin flows through the 3.4 mm (6 mesh) primary screen. This insures that only the smaller suspended solids are fed to the trickling filter. All those solids retained by the screen are shunted to the sludge storage tanks, where some anaerobic digestion is effected.

The trickling filter is designed to handle wastewater at high flow rates. The hydraulic loading applied to the trickling filter is 90,000 l/m²-day (1.5 gpm/ sq ft), including the recycle flow. The organic loading applied to the trickling filter is 14330 g/m³d (0.9 lb/cu ft-day). The recycle ratio is set at 1.5. The catch basin below the biofilter is aerated to promote bacterial action and maintain a high dissolved oxygen level in the trickling filter. Immobilized bacterial strains are used to inoculate the biofilter and are added on a monthly basis to further insure the biofilter's efficiency.

Based on pilot studies (and verified by actual data), the relevant equation to describe the trickling filter process performance is:

$$\ln \frac{C_o}{C^*} = \frac{-Kwz}{Q}$$

where

- C_o = effluent concentration, mg/l
- C_i = influent concentration, mg/l
- C* = influent concentration including recycle, mg/l = (C_i + αC_o)/(1 + α)
- α = recycle ratio = Q_r/Q_i
- Q_i = influent flowrate, m³/s
- Q_r = recycle flowrate, m³/s
- Q = total flowrate, m³/s
- w = wetter perimeter of the media, m
- z = height of trickling filter, m
- k = rate constant = 4.7 x 10⁻⁷ m/s

As can be seen, this rate constant is lower than normal (the range is from 7 x 10⁻⁷ to 3 x 10⁻⁶), which can be attributed to the higher than normal hydraulic loading. This increased irrigation rate probably exerts an excessive shear force upon the bacteria, preventing their prolonged attachment to the plastic media employed.

For the Maryland and Pennsylvania systems, a 65% reduction in COD and BOD is expected (and achieved, on the average). For the Blue Marsh system, however, the organic loading is a great deal higher (it could be as high as 50000 g/m³-day). Therefore, two stages are employed and the overall degradation per stage will range from 17 to 35 %. However, it should be noted that the influent BOD will be on the order of 850 mg/l, and the overall degradation will be 54% across the two-stage trickling filter. Most importantly, biofilters use far less energy than activated sludge systems to achieve similar levels of treatment.

From the trickling filter, the wastewater flows through the secondary vibrating screen (or the sedimen-

tation basin for the Blue Marsh system, which is identical to the primary basin for that system). The screen size in this stage is 0.15 mm (90 mesh). The screenings are sent to the sludge storage tank for further degradation, while the screened effluent proceeds to the mixed media filter.

It should be noted that while there is continuous recycle to the trickling filters, flow only proceeds to "recovery sections" when there has been influent flow. In other words, for the Pennsylvania and Maryland units, the media, carbon, and disinfection units are not used until the toilets are flushed or there has been grey water usage. The Blue Marsh facility, with its higher flows, will have all systems running continuously, except for some mid-week occasions when the equalization basin is empty.

The mixed media filter is comprised of anthracite and silica sand. The bed contains 30 cm (12 in) of 1 mm anthracite (uniformity coefficient less than 1.5) and 30 cm (12 in) of 0.35 mm sand (uniformity coefficient less than 1.22). The bed is designed to operate at 0.23 cm/s (3.4 gpm/sq ft), and is used to further remove any suspended solids from the purified wastewater.

Backwashing of the filters is effected using both air sparge and high velocity hydraulic action. The air is turned off, and the water velocity is reduced successively to 1 cm/s for 2 minutes, 0.7 cm/s for 2 minutes, and 0.3 cm/s for 2 minutes. This backwashing procedure therefore, scours the media clean and reorients the media to achieve the proper particle distribution within the bed.

Backwashing frequency is a function of the system usage. The optimum criteria for these situations is to backwash the filters when the pressure drop exceeds 10 psi. However, the recreational sites have higher usage on week-ends and holidays; obviously, then, during their periods of low flow, the biofilter sloughs a great deal of bacteria, requiring media backwash in the middle of the week. Therefore even if the pressure drop criteria is not met, filter backwash is effected at least once per 2000 gallons to insure process integrity and to minimize the need for extensive media cleaning. The pilot unit, on the other hand, which was tested in a factory environment and used more regularly, was backwashed only when the pressure differential was 10 psi.

Following mixed media filtration, the wastewater is subjected to activated carbon adsorption. There are three carbon columns, with a fourth one on standby, operated in a downflow series fashion. The flowrate is 0.23 cm/s (3.4 gpm/sq ft) and the retention time is 22 minutes for the Maryland and Pennsylvania units. While this is a short retention time, adequate performance is achieved. This is attributed to three factors. In the recycle mode, the water is continually contacting the carbon (through its multiple uses). Also, with the intermittent usage, some wastewater contacts the carbon for very long times. Moreover, bioregeneration of the bed is probably effected during these no flow periods. A 40 minute retention time and 0.11 cm/s (1.7 gpm/sq ft) velocity is employed for the Blue Marsh facility. The activated carbon columns are backwashed to remove any suspended solids, within the beds, when the pressure differential exceeds 4 psi. It is

expected that the carbon columns will function for at least one year prior to media replacement.

Once the carbon adsorption has been effected, the wastewater is disinfected. Both iodine and hypochlorite have been used as disinfectants at dosages ranging from 1 to 25 mg/l, and providing residuals of less than 1 mg/l. There has been little difference in the bacterial kill effected between the two disinfectants-- both reduce the coliform levels to below acceptable limits.

As noted in figure 1, the treated wastewater is stored for reuse in the flush water storage tank, which is connected to the backwash storage tank. This purified wastewater can then be reused for black water needs or used to clean the media and/or carbon columns. (Filter backwash water is metered to the sludge storage tank). If excess water (from grey water, fecal water, or urinal water introduction) is provided to the system, the backwash storage tank allows this water to overflow to the evaporator storage tank. That water is then evaporated, which provides heating or cooling to the comfort station module (via the evaporative cooling effect).

The sludge storage tank allows for solids settling from several streams generated by the system. Those flows fed to the sludge tank include screenings from the trickling filter and filter backwash water. When these flows enter the tank, displaced supernatant is sent to the first stage of the AWTP and processed identically to influent wastewater, thereby maximizing water recovery. Solids are held in the tank to allow digestion of the biodegradable fraction, and it is expected that pumpout will be required on no more frequently than an annual or bi-annual basis.

If land application (and not direct reuse) is to be effected (as in the Blue Marsh case), there is a slight variation in the last steps of water storage. Instead of having storage tanks for flush water, backwash, and evaporation needs, these tanks are used for backwash storage, a chlorine contact chamber, and an oxygenation basin (which are filled in the indicated series fashion). This provides for a disinfected, dissolved oxygen-rich wastewater in the last tank, which is then spray irrigated. For the Blue Marsh situation, the land application system uses 3.5 acres, divided into eight zones. The wastewater is applied at a rate of 2.6 cm/wk (1.0 in wk), five hours per day, two zones each per day. This provides a three day rest period between applications, thereby allowing the soil to assimilate the residual nutrients and diminish the probability for ponding or flooding.

SUMMARY

Therefore, these two versions of the Enviropak AWTP system provide the ultimate user a choice-- which reuse system alternative best suits his application's requirements? The AWTP reuse system's advantages include:

1. The AWTP accepts grey water, as well as black water (unlike other currently available systems).
2. The AWTP comes complete with a comfort station,

containing sinks, urinals, and toilets. Furthermore, the comfort station complies with handicap access guidelines.

3. Water is recycled only for black water needs, thereby insuring the safe non-toxic use by its patrons.
4. All excess water (which has been treated previously) is evaporated by a novel process, and provides climate control for the comfort station (cooling and heating).
5. The land application system version allows for the recharge of vital groundwater reservoirs with purified wastewater, as well as supplies needed water for irrigation purposes.

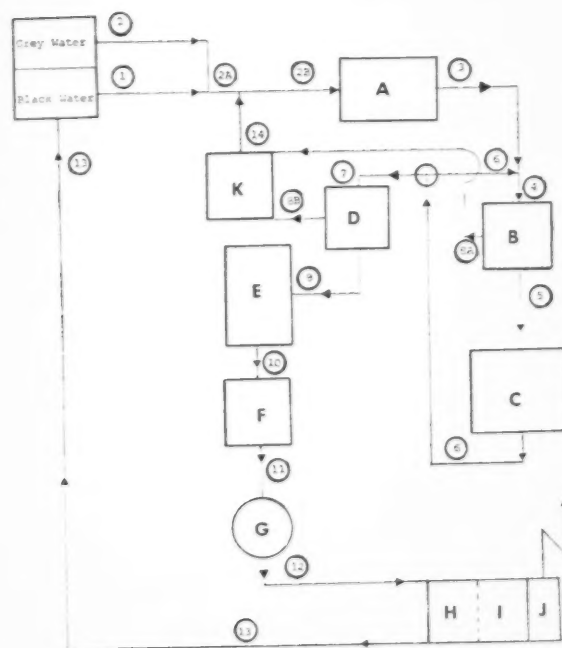
While this system is currently employed as a recreational comfort station, it can find utility for construction sites, mining facilities, as well as roadside rest areas. The Enviropak AWTP has demonstrated its utility by providing colorless, odor-less, pathogen-free flush water (or irrigation water); by accommodating the potable water employed for hand and face washing, as well as shower wastewater; and, providing climate control for the comfort station.

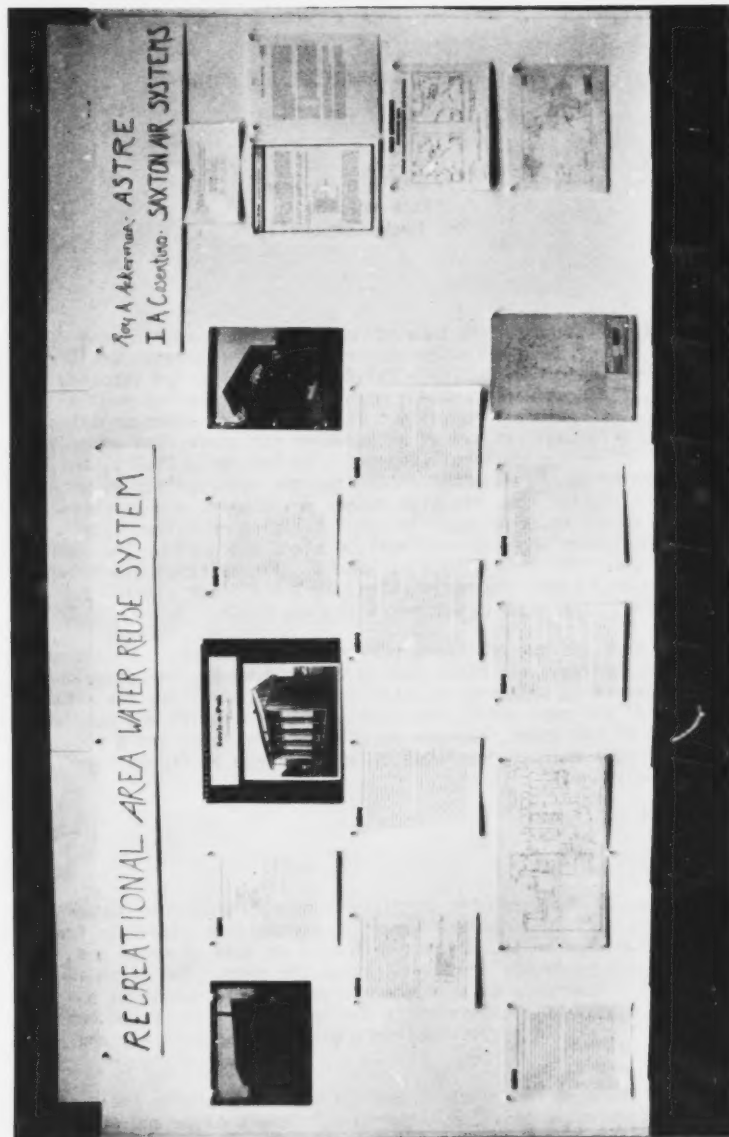
Moreover, since the system is based upon a trickling filter, energy needs are greatly reduced (since oxygen transport need not be mechanically effected as would be the case for activated sludge systems).

FIGURE 1. SYSTEM SCHEMATIC

KEY;

- A. EQUALIZATION BASIN
- B. VIBRATING SHAKER, 6 MESH SCREEN
- C. TRICKLING FILTER
- D. VIBRATING SHAKER, 90 MESH SCREEN
- E. DUAL MEDIA FILTER
- F. ACTIVATED CARBON BEDS
- G. DISINFECTION
- H. FLUSH WATER STORAGE
- I. BACKWASH STORAGE
- J. EVAPORATOR STORAGE
- K. SLUDGE STORAGE TANK





WATER REUSE AT LANCASTER, CALIFORNIA

Franklin D. Dryden
Vice President
PRC Toups Corporation

The City of Lancaster is located on the southern edge of the Mojave Desert about 80 miles northeast of Los Angeles. Until 1972 when water from northern California was imported into the valley, the area was dependent on groundwater supplies and 2-6 inches of rainfall annually. The shortage of water prompted a prohibition against use of groundwater for other than domestic, industrial or agricultural purposes. In the early 1960's, the Los Angeles County engineer recognized the desirability of water-based recreation for this high desert environment and proposed the construction of an aquatic park utilizing reclaimed water from the Lancaster water reclamation plant operated by the Sanitation Districts of Los Angeles County. The relationship between the proposed park, the reclamation plant and other significant features of the area is presented in Figure 1.

Apollo Park was opened to the public in 1972 and provided almost 150,000 user-days of recreation in 1978. However, the program faces continued problems in satisfactorily controlling the alkalinity of the park soils, the total dissolved solids and nutrient content of the lakes, mercury accumulation in fish, and a reduction of park maintenance funds as an outgrowth of Proposition 13 in California.

RESEARCH PROGRAM

Objectives

The water in the Lancaster oxidation ponds is unable to support fish life because dissolved oxygen concentrations fluctuate from supersaturation in the daytime to 1 mg/l or less at night and ammonia and pH levels are high part of the year. The dissolved oxygen fluctuation occurs as green algae utilize nutrients and produce oxygen by photosynthesis during daylight hours and bacteria deplete the oxygen by decomposing organic matter both day and night.

The objective of the research project was to determine the most cost-effective process for removing sufficient algae and algal nutrients to make the water suitable for use in recreational lakes. Preliminary review of the literature provided the basis for proposing critical quality objectives that included a low chemical oxygen demand (COD), turbidity less than 10 Jackson candle turbidity units, pH less than 8.0, few algae, and phosphate less than 2.0 mg/l as PO_4 . Subsequent experience revealed that

algae growth is not inhibited until phosphate concentrations are below 0.5 mg/l as PO_4 .

Pilot Studies

The research program was conducted in two phases. Phase I featured:

- A. Collection of basic data on the character of the water in the oxidation ponds on both an annual and diurnal basis.
- B. Jar test evaluation of possible chemical coagulants for the precipitation of phosphate and the flocculation and removal of algae and other suspended material.
- C. Pilot scale trials of applicable processes to determine basic performance and costs data.

The pilot process configurations that were evaluated are illustrated in Figure 2. Key findings of Phase I are summarized in the graphs of Figures 3 and 4. Alum was more cost-effective and absolutely effective in reducing the phosphate concentration below 1.0 mg/l. Alum was most effective at a pH slightly above 6 at any dosage.

Table 1 provides basic water quality data on the source water (Pond 3) for the pilot plants, typical pilot plant effluent, and the objectives for the full-scale tertiary treatment plant.

Phase II involved the long-term (18-month) testing of the optimum processes at a scale of about 30 gpm with simulated lakes to evaluate regrowth of algae and acceptability of the renovated water to fish. Dissolved air flotation and plain sedimentation processes were operated in parallel on alum coagulated oxidation pond water since both had performed comparably during the first phase, both were followed by either dual media filters or two stage pressure filters (also operated in parallel) prior to chlorination and discharge to a holding basin.

The research project was completed in 1966 having demonstrated that a suitable reclaimed water could be produced at costs comparable to those expected for imported water due to be delivered into the Antelope Valley about 1972. Amortized capital and operating costs were estimated at \$150 per million gallons (mg) in 1966 dollars for a 3 mgd plant.

TERTIARY TREATMENT PLANT

Design Concept

Based on the results of the research project, a 0.5 mgd tertiary treatment plant was designed by the Sanitation Districts and constructed at a total cost of \$260,000, including engineering, between May 1968 and August 1969. The tertiary plant is shown in schematic cross-section in Figure 5 and a schematic process flow diagram is shown in Figure 6.

Although both sedimentation and dissolved air flotation performed equally well in the pilot phase, sedimentation was chosen for the final design on the basis of greater simplicity and the prospect that a deeper hopper might thicken the sludge. Subsequent experience has shown that the alum sludge is not particularly susceptible to gravitational thickening.

The dual media filter was chosen over the pressure sand filter because it was the simpler design and because the pilot pressure filters showed signs of channeling and clogging when they were dismantled after testing. Design parameters for the tertiary treatment plant are presented in Table 2.

After the tertiary plant was in operation, it was modified in three minor ways:

- A. Coarse screens were added to the intake pipes in the oxidation ponds to prevent water fowl from being pulled into the piping and jamming the meters.
- B. Redwood baffles with 2-inch open spacing were installed on the overflow from the flocculation chamber to the sedimentation tank to reduce turbulence effects from the paddle wheels.
- C. The open pipe gallery was enclosed and heated to prevent freezing of unused sections of piping.

Failure of the deep hoppers to thicken the alum flocculated sludge has required a higher wasting rate than anticipated. Based on this factor, it would be recommended that air flotation be used in lieu of sedimentation for this process in the future.

Performance

Annual averages of selected operational and cost data are presented in Table 3. The unit cost has been higher than anticipated because with no overflow from the lakes, flows have been reduced in the winter time to very low levels and yearly averages are only slightly more than half their design rate.

Imported waters of potable quality are now available at \$150 per mg because amortized capital and power costs of \$450 per mg are not included in the price of water.

APOLLO PARK

Soil Reclamation

Apollo Park is located on county property adjacent to fox airfield. The 56-acre site encompasses the barrow pits created during the construction of the airport and which provided part of the inspiration for the recreational lake concept. In addition to needing a steady supply of usable water for the lakes and park irrigation, the county faced the need to reclaim the soil to permit growth of suitable park vegetation. The alkaline soil in the area was

high in sodium, salinity and boron. The soil was treated with gypsum and shaped into raised leaching fields where it was flushed with product water from the tertiary treatment plant in a total ratio of 7 parts of water for each part of soil. This treatment was only partially successful, but reduced the boron and alkalinity values to marginally acceptable levels. The soil was enriched with organic humus by mixing digested sludge and dried oxidation pond sediments from the Lancaster water reclamation plant.

After the ground surface of the park was contoured, the reclaimed soil was spread to a depth of 2 feet. Trees were planted in specially prepared holes. The three lakes have a total area of 26 acres and maximum depths of 15 feet. They are arranged as shown in Figure 7.

Mercury in Fish

Fish were first planted in the lakes in 1971. Rainbow trout, large-mouth black bass, channel catfish and red-ear sunfish thrived in the lake environment. In 14 months, rainbow trout grew from 4 - 6 inches in length up to 18 - 22 inches, and the catfish, sunfish and bass had begun reproducing. The original fishing program would not allow the fish to be eaten while the Health Department conducted a variety of analytical tests. After several years of testing the fish were declared healthy and safe in all respects except that mercury was found to have reached levels of 3 mg/kg in the trout and bass. This finding altered the fishing program to a put-and-take system after all of the original fish were killed with retanone. The program involves stocking the lake with catchable size trout, which do not reproduce and which will be caught within a few weeks to assure minimal increase in mercury levels.

Since mercury concentrations in the reclaimed water were known to be less than 0.0003 mg/l, it was determined that the mercury was being metholated into the food chain from the original lake soils. Tests of soils in the lakes and around the park indicated concentrations approaching 1 mg/kg and averaging about 0.25 mg/kg in the lake sediments. It is believed that the mercury occurring naturally in the soils in this normally dry desert environment were readily available for metholation once the soils were continuously under water. It is also theorized that some mercury is entering the lakes on windblown soils. Since the put-and-take fishing program solves the immediate problem, there are no plans to rid the lake of its mercury in the immediate future.

TDS Problems

In the original design concept, the total dissolved solids (TDS) of the lake water was expected to stabilize at about 800 mg/l, by continuously feeding in 0.5 mgd of reclaimed water with a TDS of 500-600 mg/l and withdrawing lake water for irrigation of the park and overflowing the remainder to Amargosa Creek. However, when the park was constructed, the overflow to the creek was not provided because regulatory authorities feared it would impair groundwater quality if it percolated, or breed mosquitoes

and midges if it failed to percolate into the high desert soils. The county is still attempting to obtain financing for construction of a pipeline to the normal drainage channel of the valley. In the interim, the only withdrawal has been for irrigation of the park and dissolved solids have been concentrating by evaporation. Dissolved solids reached 1400 mg/l in 1976, making the water marginal or unsuitable for irrigation of soils which are of marginal quality. Once a suitable drain is obtained, the TDS levels should be kept under control. The evaporation and concentration of dissolved solids also concentrates nutrients and thereby increases the production of both free-floating and fixed algae. If the algae is permitted to grow extensively the water may again become unsuitable for supporting fish.

ACKNOWLEDGEMENTS

The studies reported herein were conducted while the author was in charge of the Research Division of the Sanitation Districts of Los Angeles County. Portions of the work were carried out by staff members of the Sanitation District, the Los Angeles County Engineer, the U.S. Environmental Protection Agency and its predecessors as well as several other state, county and private organizations. The author would particularly like to acknowledge the contributions of project engineers Jay Kremer and John Ferguson of the Sanitation Districts, Project Officer, Gerald Stern of the EPA, and Richard E. Kahns of the County Engineer Department, Los Angeles County. Much of the research and most of the construction was funded through grants from the EPA and the State of California.

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TABLE 1

COMPARISON OF WATER QUALITY CHARACTERISTICS

CHARACTERISTICS	POND 3 WATER	PRODUCT WATER	OBJECTIVES FOR TERTIARY PLANT EFFLUENT
pH	8	6.7	6.5-8.0
Turbidity (JTU)	90	4	<5
Total alkalinity (mg/l as CaCO ₃)	260	95	<140
Hardness (mg/l as CaCO ₃)	80	100	<110
Suspended Solids (mg/l)	75	6	<10
Dissolved Solids (mg/l)	575	575	<650
Chemical Oxygen Demand (mg/l)	250	50	<75
Biological Oxygen Demand (mg/l)	38	<10	<10
Dissolved Oxygen (i.e.g/l)	0.1-40	7-15	>5
Ammonia-N (mg/l)	0.1-20	0.1-20	<1
Organic-N (mg/l)	7-20	1-3	<3
Nitrate-N (mg/l)	1-4	1-4	<4
Nitrite-N (mg/l)	0.1-12	0.1-12	<1
Total nitrogen (mg/l)	7-20	3-20	<20
Phosphate (mg/l) as PO ₄	40	0.25	<0.5
Algae (counts/ml)	200,000	7,000	<10,000
Confirmed Coliform (MPN/100 ml)	7,900	<1.8	<2.2
Chlorine Residual (mg/l)	—	0.2-0.5	<0.5
Boron			<1.4
Sodium Adsorption Ratio			<7

TABLE 2

**DESIGN AND OPERATIONAL
PARAMETERS FOR THE TERTIARY PLANT**

Parameter	Unit*	Design Value	TTP Normal Range
INFLUENT FLOW	m ³ /day (mgd)	2080 (.55)	(0.2-0.75) 760-2840
FLOCCULATION CHAMBER			
pH	pH Units	6.45	5.9-6.7
Alum. Dose	mg/l	300	225-450
Detention Time	min.	20	15-55
Paddle Tip Speed	cm/sec (ft/sec)	15.3-46 (5-1.5)	15.3-24 (5-8)
SEDIMENTATION TANK			
Overflow Rate	m/day (gpd/ft ²)	20.5 (500)	7.5-28 (182-680)
Detention Time	Hr	2.5	1.5-7
Flight Speed	cm/min (in/min)	91.5 (36)	48 (19)
Sludge Flow	% Plant Flow	5	12-15
Sludge Concentrate	% Solids, Maximum	3	0.1-75
FILTRATION			
Bed Area	m ² (sq. ft.)	17 (180)	17 (180)
Loading Rate	m/min (gpm/ft ²)	0.081 (2)	**
Final Head Loss	m (ft)	2.14 (7)	2.14 (7)
Max. Backwash Cycle	Hr.	24	24
Max. Backwash Rate	m/min. (gpm/ft ²)	0.73 (18)	**
Filter Backwash Waste	m ³ /day (mgd)	76 (.02)	57-76 (0.15-0.20)
Surface Wash	m/min \$3.5 kg/cm ² (gpm/ft ² @ 50 psi)	0.081 (2)	0.081 (2)
Bed Expansion	%	50	50
CHLORINATION			
Influent Dose	mg/l	0-15	3.5-16.0
Effluent Dose	mg/l	0-15	4.5-19.0
Contact Pond Detention	Hr	8-10	7-24
FINAL EFFLUENT			(.15-.65)
Flow	m ³ /day (mgd)	1900 (.50)	570-2460
Phosphate	mg/l	0.50	0.10-0.40
Turbidity	JTU	5	0.08-3.0
PLANT EFFICIENCY	% of infl. flow	91	77.86
ALUM COST @ \$55.60/Ton	\$ per 1000m ³ eff. (\$ per MG eff.)	20.40 (77)	18.29 (68-110)

*English units are shown in parenthesis
**Indicates no data available

TABLE 3
ANTELOPE VALLEY TERTIARY TREATMENT PLANT
AND
APOLLO LAKES
EFFLUENT AND OPERATIONAL DATA

CALENDAR YEAR	TERTIARY TREATMENT PLANT DATA							O & M COSTS		APOLLO LAKES TDS ^{c, d} (mg/l)
	Effluent Delivered Annually (mg)	EFFLUENT QUALITY			Alum ^a Dosage (mg/l)	Total ^{a, b} Phosphate (mg/l)	TDS ^c (mg/l)	Total (Dollars)	_____(Dollars)_____ (MG)	
		Turbidity ^a (JTU)	TDS ^c	Total ^{a, b} Phosphate						
1971	125.	1.5	552	333	0.31	333	30,900	247	--	
1972	140.	1.5	532	328	0.24	328	55,500	396	910	
1973	115.	1.4	541	305	0.23	305	52,100	453	--	
1974	109.	1.2	517	339	0.24	339	36,300	333	--	
1975	102.	0.9	514	357	0.26	357	53,100	521	--	
1976	93.	1.3	644	352	0.26	352	50,000	537	1,429	
1977	96.	1.1	543	352	0.24	352	50,700	528	1,377	
1978	96.	1.1	498	311	0.21	311	48,800	456	1,167	

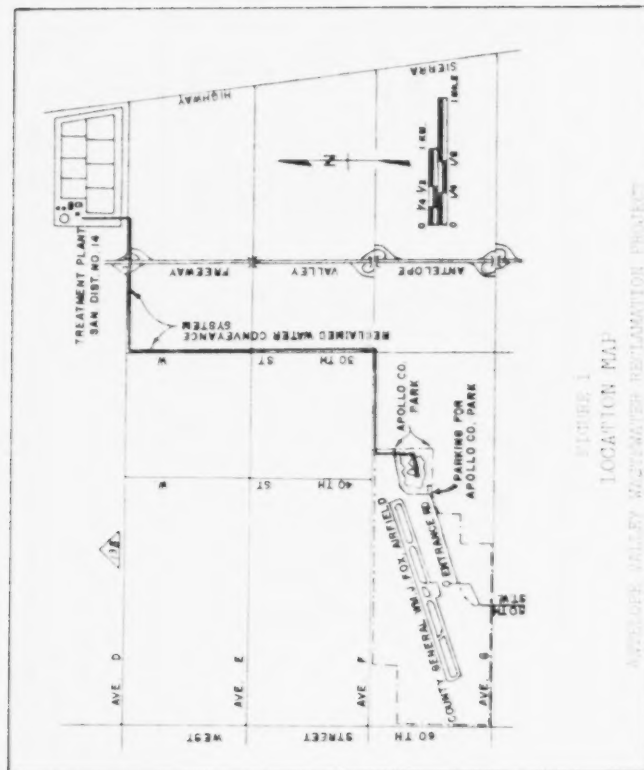
a. Measured daily.

b. Measured as orthophosphate

c. Measured semi-annually

d. Average for three lakes

e. Capital costs of \$260,000 amortized 30 yrs. at 7% interest — \$116.00/mg at 0.5 mgd.



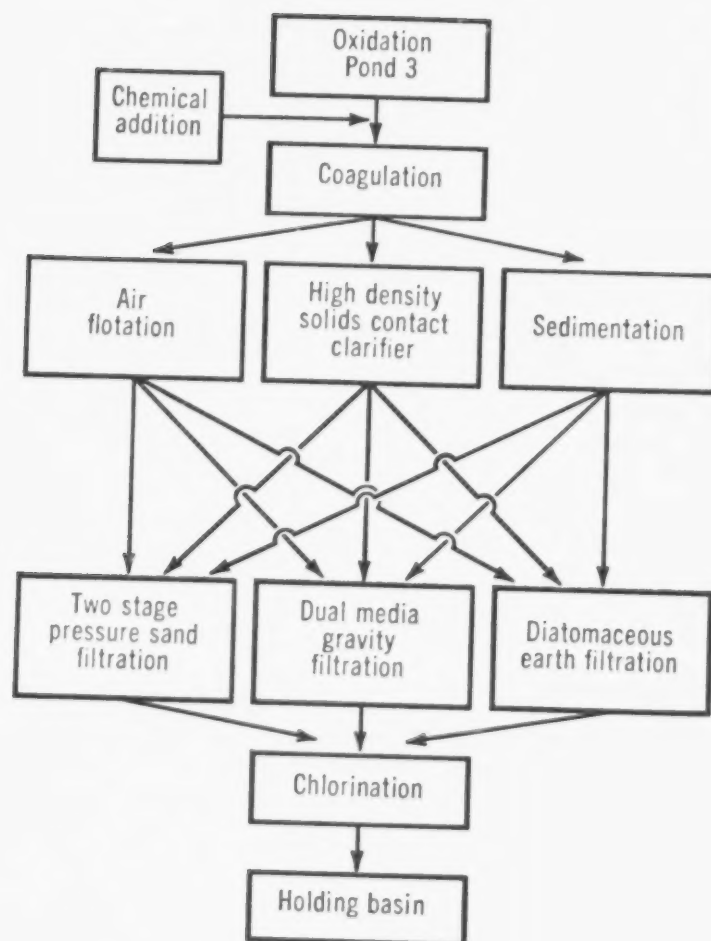


FIGURE 2
PILOT PLANT PROCESSES AND SEQUENCES EVALUATED BETWEEN
1964-1966

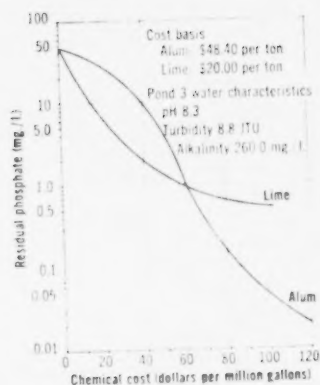


FIGURE 3
 Residual phosphate for
 alum and lime coagulation
 based on jar tests

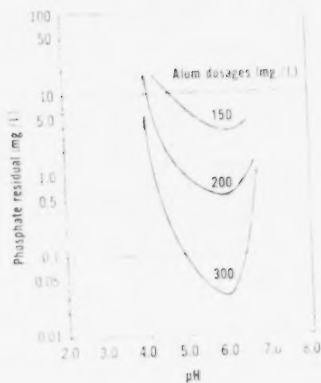


FIGURE 4
 Phosphate residual vs.
 pH based on jar tests

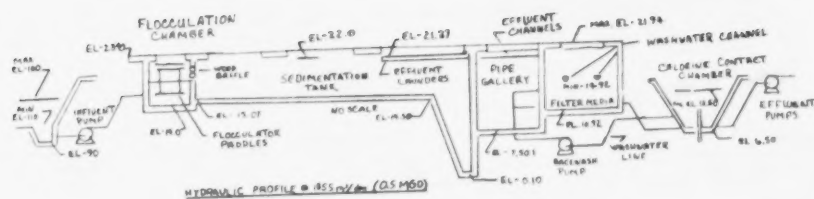


FIGURE 5
 TERTIARY TREATMENT PLANT HYDRAULIC PROFILE

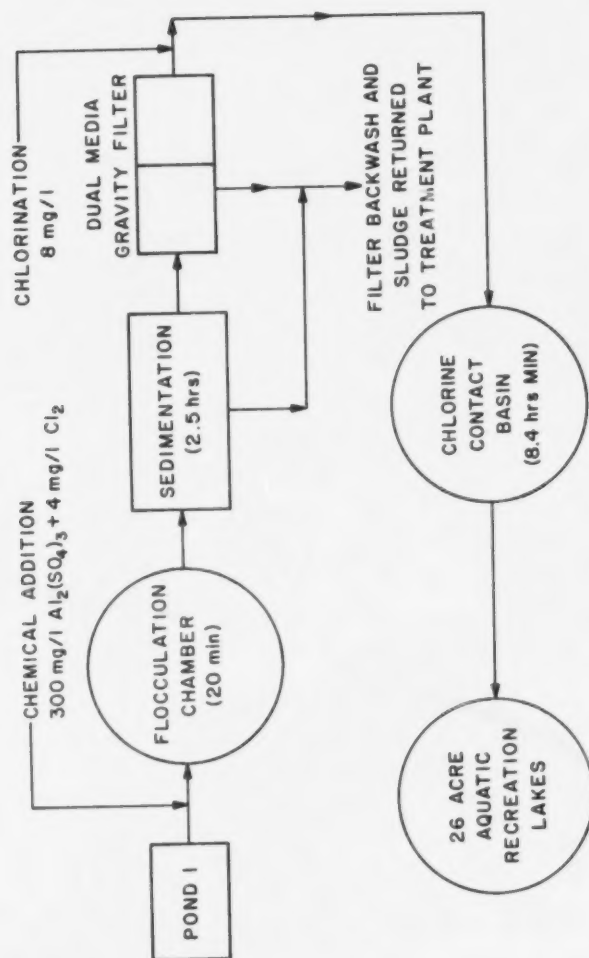


FIGURE 6
LANCASTER TERTIARY TREATMENT PLANT
PROCESS FLOW DIAGRAM

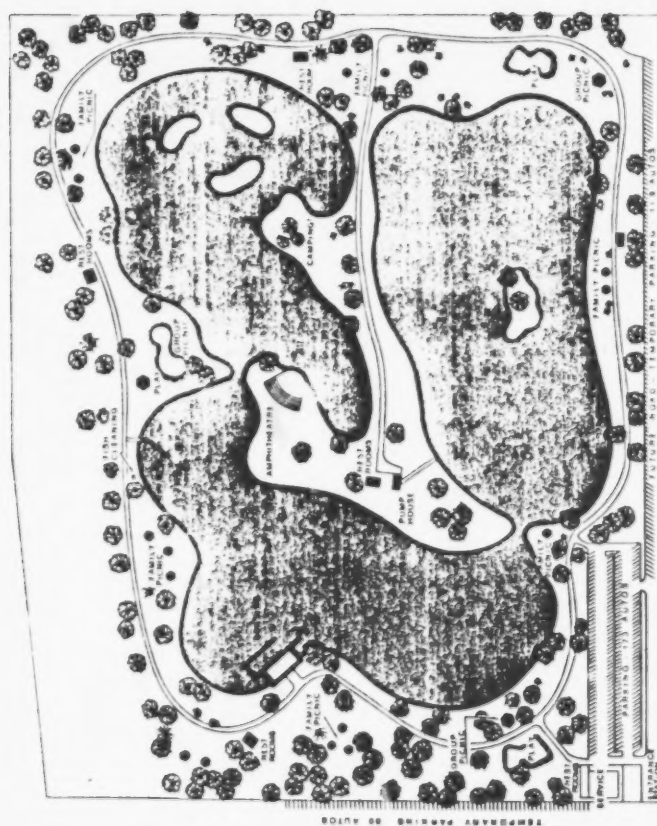
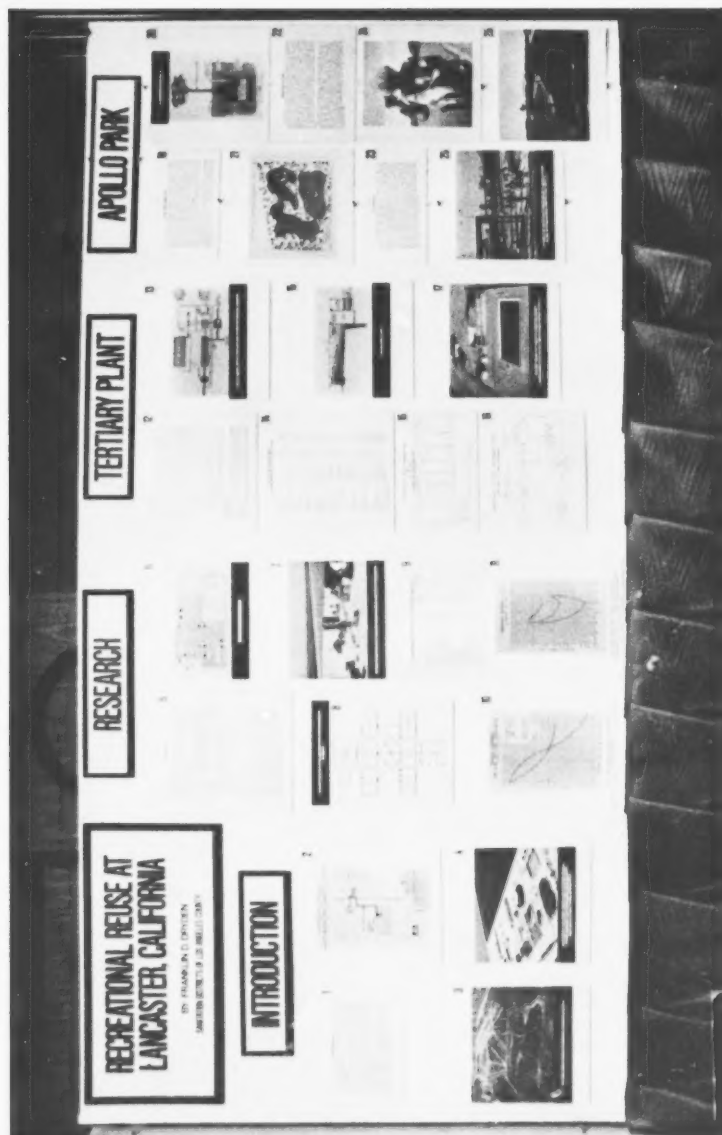


FIGURE 7
PLAN LAYOUT OF APOLLO PARK





MODELING, MONITORING AND INSTRUMENTATION

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ON-LINE AUTOMATED WATER QUALITY MONITORING

Kenji Nishioka
Project Manager, Ames Research Center
Moffett Field, CA 94035

William Roman
Research Engineer, Santa Clara Valley Water District
San Jose, CA 95118

Richard Brooks, Operations Engineer
David Nibley, Microbiologist
Richard Thomas, Chemist
The Boeing Company, Houston, TX 77058

INTRODUCTION

NASA entered into a joint experiment with the Environmental Protection Agency (EPA) and the Department of Housing and Urban Development (HUD) to apply space technology in developing a concept and basic guidelines for an urban housing complex designed for total recycling of waste products.⁽¹⁾ This effort was called the Modular Integrated Utility System (MIUS) project. It was determined that there is a definite need for continuously monitoring water quality in recycling wastewater. A workshop was convened to determine which parameters should be monitored.⁽²⁾ Commercially available instruments were surveyed and a conceptual design of an automated water quality monitoring system (WQMS) was initiated.

The sensors necessary for water quality monitoring included coliform, total and viable biomass, viruses, and halogenated hydrocarbons. Such sensors were not available but have subsequently been developed or are being developed based on research at several NASA centers. Upon completion of the WQMS conceptual design, the need for and possible interest in such a system by the wastewater community was surveyed.⁽³⁾ Results from the survey indicated a definite need for technology development in this area. Based on this finding, NASA has designed, assembled, and is field-testing an automated water quality monitoring system. The system underwent preliminary checkout at the Southwest Wastewater Treatment Plant in Houston, Texas, from March through May 1977. In June 1977, the system was moved to Palo Alto, California, to start a joint demonstration test with the Santa Clara Valley Water District (SCVWD). The system is currently being used to monitor the quality of water produced by the SCVWD experimental wastewater reclamation facility.⁽⁴⁾ This facility, built with help from EPA and the State of California, uses the output from the city of Palo Alto's secondary wastewater treatment plant as input water to be reclaimed.

The objectives of the project are to demonstrate the feasibility of automated water quality monitoring and to evaluate overall concept technology readiness, including selected commercial sensors, NASA-developed sensors, and the computer system. This paper describes the overall system and the specific sensors NASA has developed. Some preliminary program results, their implications, and problems encountered are also discussed.

DESCRIPTION OF SYSTEM

The WQMS is essentially an automated water quality laboratory housed in an 8 × 30 ft (2.4 × 9.0 m) converted surplus radar van. Most functions associated with this laboratory system, namely, sampling, sensor calibration and control, data acquisition, analysis, storage, and report generation, are computer controlled and automated. Figure 1 is a schematic of the overall system and its operation. As shown, the system can logically be divided into four major subsystems: sample collection and distribution, sensor, computer, and general support.

Seventeen parameters can be currently monitored (the halogenated hydrocarbons are treated as a single parameter). However, due to the low levels of nitrate present (1.0 ppm; it is not monitored), only 16 parameters are currently monitored (see Table 1). With the six different sample lines and a 2-hr recycle time per sample line for most sensors (except for the coliform sensor), over 180 data points are collected and stored every 24 hours. Up to 93 days of data (including the current day) are stored on the disk (see section on "Computer Subsystem") and can be easily accessed for reference or printing. Data are transferred from the disk to magnetic tape for archival storage.

Sample Subsystem

As configured, the system can handle up to six different incoming sample lines. These sample lines are routed from various points in the reclamation plant and provide process stream samples for monitoring the effectiveness of the various SCVWD plant treatment processes. Figure 2, a schematic of the SCVWD reclamation plant, shows each sample point from which a continuous sample stream is piped to the automated laboratory. The computer (either on a preprogrammed or instructed mode) selects one of the sample lines to feed to the sample conditioning system (Fig. 3). The entire operation, from valve opening to backflushing of the lines, is automated. As shown in the schematic, the entire subsystem is redundant to prevent system downtime due to clogged filters (which occurs even with backflushing) and component failure from the constant hammering of the backflush piston. The sample stream is pumped to the sensors in the unfiltered and filtered sample circuits. The unfiltered sample is routed to the total oxygen demand (TOD), total organic carbon (TOC), turbidity, and dissolved oxygen (DO) sensors. Filtered samples are supplied to the pH, chloride, ammonia, nitrate, conductivity, temperature, residual chlorine, hardness, biosensor, total coliform, and gas chromatograph (halogenated hydrocarbons) sensors. Piping and tubing used in the sample acquisition and distribution subsystem are principally polyvinyl chloride (PVC). There was concern that the PVC would affect the sample analyses, particularly in the gas chromatograph analyses, but such an effect has not been noted.

Sensor Subsystem

The sensors used in the automated laboratory are either commercial sensors or NASA-developed sensors. With the exception of the NASA sensors, which are the biosensor, coliform, and gas chromatograph, all parameters shown in Figs. 1 and 3 are measured by commercial sensors bought either from commercial suppliers or from government surplus. The sensor packages were modified to make them compatible with automated computer-controlled operation.

The three NASA-developed sensors are based on technologies partially or totally developed in the space research program. The biosensor,

coliform, and gas chromatograph sensors are installed in the trailer laboratory and are part of the demonstration test.

Since details on these three sensors are not available elsewhere, they are described below.

Coliform sensor. The automated coliform sensor is based on a method similar to the Standard Methods Most Probable Number (MPN) Test. The sensor employs redox electrodes to monitor changes in the coliform specific media when bacterial growth takes place. The time required for the electrodes to detect a change in the media can be directly related to the initial concentration of coliform bacteria in the sample. In addition, the differentiation of total and fecal coliform can be achieved by using different incubation temperatures. The entire operation, with the exception of sample collection, is automated and includes sample injection, incubation, detection, analysis, and sensor clean-up. When the clean-up cycle is completed, the sensor is ready for the next test.

The coliform sensor consists of eight individual thermostatically controlled incubation cells. The temperatures are maintained at 37° or 44.5°C for monitoring total or fecal coliforms, respectively. The cycle is initiated when a sample of 14 ml is automatically injected into each cell containing 14 ml of lauryl sulfate broth. The redox potential of the media is continually monitored as the coliform bacteria grow and multiply. Calculations of the initial coliform concentration is based on the time required to achieve a 200-mV change in the electrode output. After all the cells have reacted or the maximum reaction time of 14 hr has elapsed, the sensor is automatically prepared for another analysis. The cells are first drained of spent media and flushed with 0.1N nitric acid to sterilize and clean the platinum electrodes. The cells are then flushed with 95°C water and drained. The cells are finally refilled with 14 ml of lauryl sulfate broth, heated to 85°C for a final sterilization, and cooled to the appropriate temperature for the next incubation. The complete cycle of inoculation, incubation, analysis, sterilization, and introduction of fresh media is completed once each 24-hr period. Currently, all eight cells are being inoculated with the same sample; however, it is theoretically possible to inoculate each cell with a different sample, thereby permitting an analysis once every 3 hr. Current sensitivity is about seven bacteria per 100 ml, but this can probably be improved with larger sample sizes or through higher concentration of bacteria.

Biosensor, luminol-carbon monoxide. A chemiluminescent method for monitoring bacteria has also been developed for the WMS. The system employs the alkaline luminol-hydrogen peroxide reaction with bacterial iron porphyrins to quantify bacteria. A combination of techniques, including hydrogen peroxide pretreatment, reaction rate differentiation, and ethylene diamine tetracetic acid (EDTA) provides the specificity for monitoring bacteria in wastewater effluents. In addition, a new technique using carbon monoxide gas to react with the iron porphyrins has been developed which permits differentiation between viable and dead organisms. The CO reaction with the iron porphyrins in the live (viable) bacteria results in the suppression of the chemiluminescence action, and thus the luminescence will be from the dead bacteria only. Therefore, with the total and dead bacteria known, the viable bacteria can be determined.

The chemiluminescent biosensor is a rapid method for monitoring bacteria in wastewater. Within 2 hr, both total and viable bacteria

measurements can be obtained. While this sensor is not specific for a particular organism, the results can be used as an indicator of the biological quality of the water.

Gas chromatograph. A computer-controlled automated twin-gas chromatograph (GC) has been developed which monitors nine volatile halocarbons within a 50-min period. The GC consists of an automatic sampler that injects a water sample into a preparative GC for separation of the organic fraction from the water. The organics collected in a trap are then heated to introduce the compounds into the analytical GC for detection by an electron capture detector (ECD). The instrument is capable of quantitative analysis at the part per billion level.

Computer Subsystem

The computer subsystem is the key to the concept of automated water quality monitoring. Two Data General Corporation minicomputers, a NOVA 1200 and NOVA 3D, are used to continuously monitor the chemistry and biology of wastewater at various process stages within the treatment facility. A Monitor Labs Model 9400 Data Logger and a Monitor Labs Model 1300 Scanner/ADC provide the analog-to-digital sensor signal conversions. The NOVA 1200 computer system is used for data acquisition, standardization, and calibration of commercial sensors. It is referred to as the ADAM System (air data acquisition and monitoring). The NOVA 3D computer system generates the reports and is referred to as the EVE system (environmental verification and evaluation). ADAM, an independent system, operates and controls the commercial sensors and passes the chemical data from these sensors to EVE. EVE utilizes the Data General Corporation Real-Time Disk Operating System to accept the ADAM data and the data from the biological and gas chromatograph sensors it operates and controls. The data are stored, analyzed, displayed, and formatted into various reports.

Command keyboards with cathode-ray tube displays are provided to communicate with the computer and also to display current and stored information. Hard copies of the EVE reports and plots are printed using a Versatec line printer. The computer system is also equipped with both a paper tape punch read/write device and a magnetic tape read/write device for loading the operating system programs, storing data from the operating disk system, or for updating the program.

General Support Subsystem

The trailer-laboratory is self-contained, but electrical power, tap water, and analytical gases must be supplied. The trailer is equipped with a 30-kVA power transformer requiring a 440 three-phase 50 A (maximum) per phase input. This provides the necessary 220 and 110 V for the basic trailer support equipment such as air conditioning, computers, and instrumentation. Tap water supplied to the trailer is treated by a Millipore system so that the analytical instruments requiring water are supplied with filtered deionized water.

Analytical gases required are CO gas for the biosensor and a 5% methane-in-argon gas and pure nitrogen for the GC. Because some of these gases are toxic and flammable, gas detectors are provided in the trailer laboratory.

RESULTS

The results of interest encompass the overall operation of the system, the performance of the equipment, and the detailed sensor data obtained. Because this demonstration project is not yet complete, only partial results are available. Those presented here were screened, and only valid and representative examples of the WQMS and SCVWD experimental water reclamation facility are included. The automated WQMS results are periodically validated by standard wet chemistry analyses by SCVWD Reclamation and Palo Alto Wastewater Facilities. The GC results have been validated by a comparable laboratory unit at Ames Research Center and by SCVWD through their arrangements with Stanford University and Hewlett-Packard Corporation.

The overall system is operating and performing very well as a unit. Some individual instruments are not performing as expected, and there have been intermittent equipment failures. However, based on the results to date, the concept appears feasible.

The computerized data system provides total system information accessibility by means of the on-line CRT display or Versatec printer. Overall system status can be easily accessed. For example, the current status of the coliform sensor with its 95 analytical steps can be displayed quickly. Also, the detected values over the last hour shown in 15-min intervals can be displayed for each sensor. This allows the facility operator to monitor the dynamics of the plant. For example, wide swings in the detected values would indicate that the quality of the facility influent (see Fig. 4)(5) is varying significantly or that the treatment processes are not operating satisfactorily (see Fig. 5).(5)

Tables 1 and 2 and Figs. 4 and 5 show examples of actual facility operating data. Before discussing these data, it is important to note that the SCVWD Reclamation Facility is an experimental plant designed for flexibility and thus is capable of being operated in many configurations; that is, the combinations of treatment for the influent wastewater can be varied in many ways. The treatment processes that the facility is capable of conducting singularly or in any combination are:

- a. Lime (floculate and clarify)
- b. Ammonia stripping
- c. Recarbonation
- d. Ozonation
- e. Chlorination (at four points: after ozonation, after filtration, after activated carbon, and at storage tank)
- f. Polymer (at two places: after lime and after ozonation)
- g. Sand filter
- h. Activated carbon

With these many possible variations, the resulting quality of the effluent water can be expected to fluctuate. This is shown in Fig. 4 where monthly data for turbidity are presented. The top graph shows the daily average values as a solid line with $\pm 1\sigma$ variations indicated by the dotted lines. The lower graph shows the peak 1-hr average values for the sensor. The time of day is shown at the top of the lower graph. When the standard deviation and peak value curves are examined in concert, a good indication of the daily and monthly fluctuation in water quality can be obtained. For example, on March 16, 1978, the standard deviation was very high, about 100% of the average daily data value, and signified a wide fluctuation in the water quality.

Also, for March 20, the relatively high average hourly peak accompanied the high standard deviation for the day and confirmed that the high standard deviation was correct. Also, if the facility effluent (Fig. 4) is compared to the plant influent (Fig. 5), a quick evaluation of facility performance for turbidity can be made.

Figure 6 shows a daily plot for total halocarbons, TOC and turbidity for December 13, 1978. These data indicate that correlations between these parameters exist, especially for secondary effluent samples. Computer analyses of the data are continuing to determine whether these and similar correlations exist between the various parameters.

The weekly and daily computerized data results are shown in Tables 1 and 2 (from Ref. 5). Table 1 shows the weekly summary for the SCVWD reclamation facility effluent (similar summaries for the other five sample lines, although not included here, are available). The example daily data summary is shown in Table 2. The computer displays either the "influent" or "effluent" columns only, but the table was structured to illustrate how the daily data for the reclamation facility input (influent) and output (effluent) can be compared to determine how effectively the facility operated in reducing the various contaminants in the water. When these types of data are accumulated over a period of time, the facility processes can be evaluated, which is one of the objectives of the demonstration.

Results — NASA-Developed Sensors

The automated coliform sensor provides a rapid method for quantifying coliform bacteria in wastewater samples. For seeded samples there has been an excellent correlation (0.96) between the coliform sensor results and the Standard Method Most Probable Number test for total coliform. In this comparison the coliform values used were the mean of the eight cells inoculated with the split sample. The advantages of the coliform sensor are (1) shorter detection time and (2) total automation and calculation. The reaction time for the coliform sensor ranges from 7 hr for 10^5 coliform per 100 ml to 11 hr for 10 coliform per 100 ml. This is a significant improvement in detection time over other methods for coliform detection. As stated earlier, the coliform sensor is completely automated.

The accuracy of the chemiluminescent biosensor has been checked by using a Coulter counter to determine numbers of bacteria-sized particles as an indication of total bacteria. Measurements of metabolite and adenosine triphosphate (ATP), ubiquitous to viable bacteria, were also made to check the sensor readings.

The chemiluminescent biosensor measurement of total bacteria versus particle counts from a Coulter electronic particle counter showed a correlation coefficient of 0.8952 for 10 samples — a significant correlation. The correlation coefficient for the viable bacteria measurements versus the ATP analysis was 0.8625.

Since the chemiluminescent biosensor measures many different species of bacteria, the differences in the results probably reflect differences in the iron porphyrin levels among various bacteria. The biosensor-inferred measurement for live bacteria includes all metabolically active bacteria, not merely bacteria capable of division as in classical laboratory measurements. If these factors are considered, the results from the biosensor can be used as a measure of the biological quality

of the water. The biosensor offers the advantage of a rapid method (not commonly available in microbiology) for gross bacterial assessment, both total and viable.

As mentioned earlier, the automated twin-column gas chromatograph monitors nine volatile halocarbons which include the common solvents methylene chloride, 1,1,1-trichloroethane, trichloroethylene, and tetrachloroethylene; the trihalomethanes chloroform, bromodichloromethane, dibromochloromethane; and bromoform and 1,2di-chloroethylene. The sum of the concentrations for these compounds can be calculated and plotted by the computer as illustrated in Fig. 7. This figure depicts the daily averages for the secondary effluent sample from the City of Palo Alto Facility for August 1978. Data displayed in this manner permit visualization of the weekly cyclic nature of the organic levels of the water passing through the treatment plant. It is interesting to note that halocarbon levels are particularly low on weekends, but increase progressively throughout the week and possibly reflect local work patterns for the surrounding industry.

Concentrations of total halocarbons for the same water after being fully processed are illustrated in Fig. 8. Monitoring the reclamation plant effluent has shown that nominally more than 90% of the halocarbons are removed from the influent water by the various treatment processes. Intermediate sampling points in the treatment process show the effectiveness of the individual plant process. Figure 9 illustrates the effects on one contaminant chloroform when 25% of the activated carbon in the carbon tower was regenerated. An almost instantaneous 80% reduction in chloroform concentration was observed. After nine days the chloroform levels returned to their original concentrations, indicating possibly that the effectiveness of the regenerated carbon has been depleted. Another example is shown in Fig. 10 for the ammonia stripping process whereby the aerators were put on-line June 20, 1978, and a 60% reduction in chloroform concentration was measured.

Continuous unattended operation of the GC monitors the process streams to detect fluctuations of volatile halogenated hydrocarbons at various points in the treatment process. The rapid method permits monitoring the quickly changing character of water quality such as illustrated above. Use of this information permits plant operators to control and optimize plant processes.

The WQMS computer system can provide the water quality data in graphical form as shown in the previous discussion of the GC data or in tabular formats. As already stated, instantaneous displays are available and provide information for all sensors, including 15-min, 30-min, hourly, and daily averages.

The sample results presented above highlight the kinds of information that can be expected with an WQMS-type system.

SHORTCOMINGS OF THE SYSTEM AND TYPICAL PROBLEM AREAS

The project has had its share of difficulties with both hardware and software. For example, for the computer system, the original EVE computer suffered from intermittent core memory problems and software problems caused by overheating and insufficient memory, respectively. Disk space was also inadequate when multitask programs were used. The EVE system was upgraded in May 1978 to a Data General NOVA 3D (64K memory) Phoenix disk drive (10 megabytes) and a communication chassis. This system suffered from intermittent failures for 6 months before the cause was finally traced to a combination of signal noise

on the data bus and conflicting priorities between the multitask programs. All hardware and software problems have now been resolved and the system is operating.

Problems have also been encountered with the commercial sensors. These have included failures of mechanical parts and electrical components and the incompatibility of one sensor's assembly glue with its required reagent. In many cases, the overall sensor reliability was poor, but in fairness to the manufacturers it should be noted that most of these sensors are at least four years old and not designed for continuous automated operation. It is believed that many of these sensors have been upgraded by the manufacturers during this period.

Sensors Developed by NASA/Boeing and NASA/TRW

Coliform sensor. Problems with this sensor were expected since this is a prototype unit. One problem is that coliform residuals remaining in the sample line from previous samples contaminate the new samples. Another major problem is cleansing the incubator system between samples to remove residuals that would bias the next sample. These problems were resolved with the development of clean-up procedures and techniques. Some minor electronic problems with heating element thermistors were also encountered.

Biosensor, luminol-carbon monoxide. This sensor, the prototype automated chemiluminescence sensor, has operated very well. As expected with a prototype, there have been problems, the most significant being false signals. These problems were due primarily to two factors: (1) background interference resulting in erroneously high readings and (2) bacteria clumping together and producing nonhomogeneous samples. A second major problem was converting the carbon monoxide laboratory method to a fully automated operation. Both problems have been resolved satisfactorily.

Gas chromatography. The problems encountered were primarily with procedures for the preparation and use of standard solutions for calibrating the GC. Extensive collaboration between the Project, the Life Sciences Group at Ames Research Center, and the Civil Engineering Laboratory at Stanford University resulted in defining the necessary procedures. Based on these procedures, many solvents were screened until a solvent (glacial acetic acid) with an acceptable reproducibility index of 15% was found. Currently, this solvent is used in the standard solution for calibrating the GC.

FUTURE

Preliminary results from the project indicate that the concept of on-line Automated Water Quality Monitoring is feasible. However, commercial sensors require further development to enhance their reliability, service lifetimes, and ease of maintenance for continuous automated operation. The sensors developed specifically for the WQMS trailer require further development and packaging into smaller units.

Other key sensors, such as a viral detector and a phenol sensor, are being developed and, if they prove feasible, will greatly enhance the system. Several factors show the desirability of the type of technology contained in the WQMS: first, a decrease in the availability of pristine water coupled with an increase in water demand; second, the increasing contamination of water supplies coupled with an increasing number of contaminants; and, third, an increase in the use of

recycled water. Automated water quality monitoring provides the necessary evaluation of data to make the concept of totally automated water treatment a realizable goal. The ever-increasing analytical demand for water of assured quality requires that the treatment process be more comprehensive and thus more expensive. All this implies not only monitoring but also integrating measurements of water quality with water treatment in a closed-loop control. Systems that accept water quality data can optimize treatment processing and ensure that the outflow meets water quality criteria with minimum use of chemicals and energy, thereby avoiding expensive holding tanks and retreatment for water that does not meet specifications.

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Table 1. Weekly Data Summary: SCWWD Reclamation Plant Effluent;
March 18-24, 1978

SENSORS ^a	UNITS	3/18/78 SAT	3/19/78 SUN	3/20/78 MON	3/21/78 TUES	3/22/78 WED	3/23/78 THURS	3/24/78 FRI
RESIDUAL CHLORINE	mg/l	2.0	0.8	1.2	1.4	2.2	1.9	1.1
TURBIDITY - SO ₂	mg/l	1.4	0.3	6.0	2.7	2.9	3.1	2.5
TOTAL OXYGEN DEMAND	mg/l	18.6	13.9	11.5	12.8	13.7	14.6	21.5
TOTAL ORGANIC CARBON	mg/l	14.5	6.7	8.1	13.7	17.4	18.9	17.4
AMMONIA	mg/l	6.8	6.8	6.8	6.9	6.8	6.6	6.6
pH	pH	1504.0	1520.0	1506.0	1457.0	1463.0	1457.0	1442.0
CONDUCTIVITY	µmho/cm	8.9	8.2	8.0	8.1	9.4	8.9	9.0
DISSOLVED OXYGEN	mg/l	-	-	-	27.7	14.5	20.4	32.2
GAS CHROMATOGRAPH	ppb	-	-	-	0.1	0.1	0.1	0.1
METHYLENE CHLORIDE	-	-	-	-	7.8	9.5	7.3	11.1
DICHLOROETHYLENE	-	-	-	-	1.9	3.0	2.6	2.9
CHLOROFORM	-	-	-	0.8	0.2	0.4	0.4	0.2
TRICHLOROETHANE	-	-	-	-	5.8	9.3	10.0	8.9
BROMODICHLOROMETHANE	-	-	-	-	2.6	4.3	2.7	3.8
TRICHLOROETHYLENE	-	-	-	-	-	-	-	-
TETRACHLOROETHYLENE	-	-	-	-	-	-	-	-
BIOSENSOR	-	-	-	-	-	-	-	-
TOTAL BIOMASS	10 ⁶ CELLS/ml	8.7	7.0	-	6.7	7.4	-	10.6
VIABLE BIOMASS	10 ⁶ CELLS/ml	4.0	1.9	-	2.2	4.6	-	-
TEMPERATURE (SAMPLE)	F	71.0	70.0	70.0	70.0	70.0	69.0	68.0
SODIUM ^b	mg/l	108.0	110.0	122.0	130.0	121.0	113.0	125.0
CHLORIDE ^b	mg/l	-	-	265.0	248.0	304.0	300.0	294.0
COLIFORM ^c	CELLS/100 ml	7 ^d	-	-	7 ^e	7 ^f	-	73 ^g

^aNITRATE WAS FOUND TO BE ON THE ORDER OF 1.10 ppm AND IS NOT MONITORED. TOTAL NITROGEN AND
HARDNESS SENSORS HAVE NOT OPERATED RELIABLY AND WILL BE REPLACED.

^bDATA FROM WEEK JANUARY 14 20, 1978.

^cDATA FROM DATES SHOWN.

^d4/15/78.

^e4/18/78.

^f4/19/78.

^g4/14/78.

Table 2. Daily Data: SCWWD Reclamation Plant; March 24, 1978

SENSORS 3/24/78	UNITS	INFLUENT VALUE	EFFLUENT VALUE	PERCENT REDUCTION
RESIDUAL CHLORINE	mg/l	9.2	1.1	-
TURBIDITY - SiO ₂	mg/l	22.4	2.5	89
TOTAL OXYGEN DEMAND	mg/l	(INSTRUMENT DRIFTS - DATA NOT RELIABLE)		
TOTAL ORGANIC CARBON	mg/l	32.0	21.5	33
AMMONIA	mg/l	23.2	17.4	25
pH	pH	6.8	6.6	-
CONDUCTIVITY	µmho/cm	1481.0	1442.0	3
DISSOLVED OXYGEN	mg/l	4.0	9.0	125 (INCREASE)
GAS CHROMATOGRAPH	ppb	99.0	32.0	68
METHYLENE CHLORIDE		<0.1	<0.1	-
DICHLOROETHYLENE		84.9	11.1	87
CHLOROFORM		148.8	2.9	98
TRICHLOROETHANE		6.3	0.2	97
BROMODICHLOROMETHANE		456.1	8.9	98
TRICHLOROETHYLENE		208.3	3.8	98
TETRACHLOROETHYLENE		73.0	68.0	
TEMPERATURE	F			
BIOSENSOR ^a	10 ⁶ CELLS/ml			
TOTAL BIOMASS		47.8	7.4	85
VIABLE BIOMASS		8.1	4.6	43
TOTAL COLIFORM ^b	CELLS/100 ml		73.0	-
SODIUM ^c	mg/l	150.0	125.0	17
CHLORIDE ^c	mg/l	182.0	294.0	62 (INCREASE)

^a DATA FROM 3/22/78.

^b DATA FROM 4/14/78.

^c DATA FROM 1/20/78.

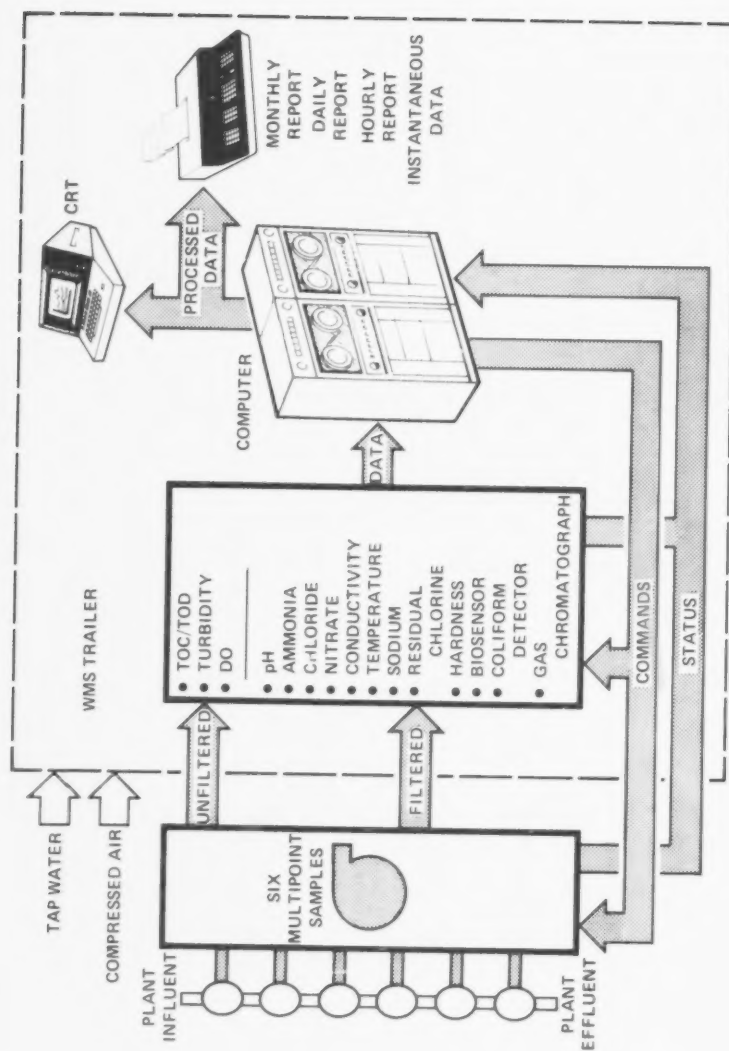
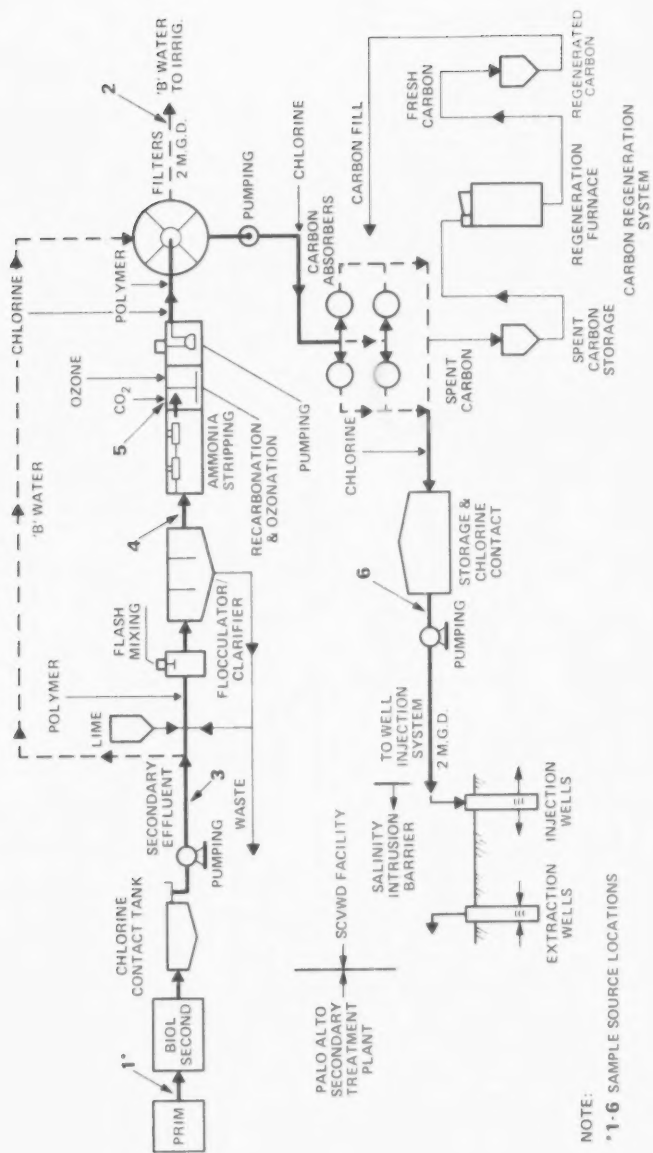


Figure 1. Water Monitoring System.



NOTE:
*1-6 SAMPLE SOURCE LOCATIONS

Figure 2. Flow Diagram of SCVWD Facility.

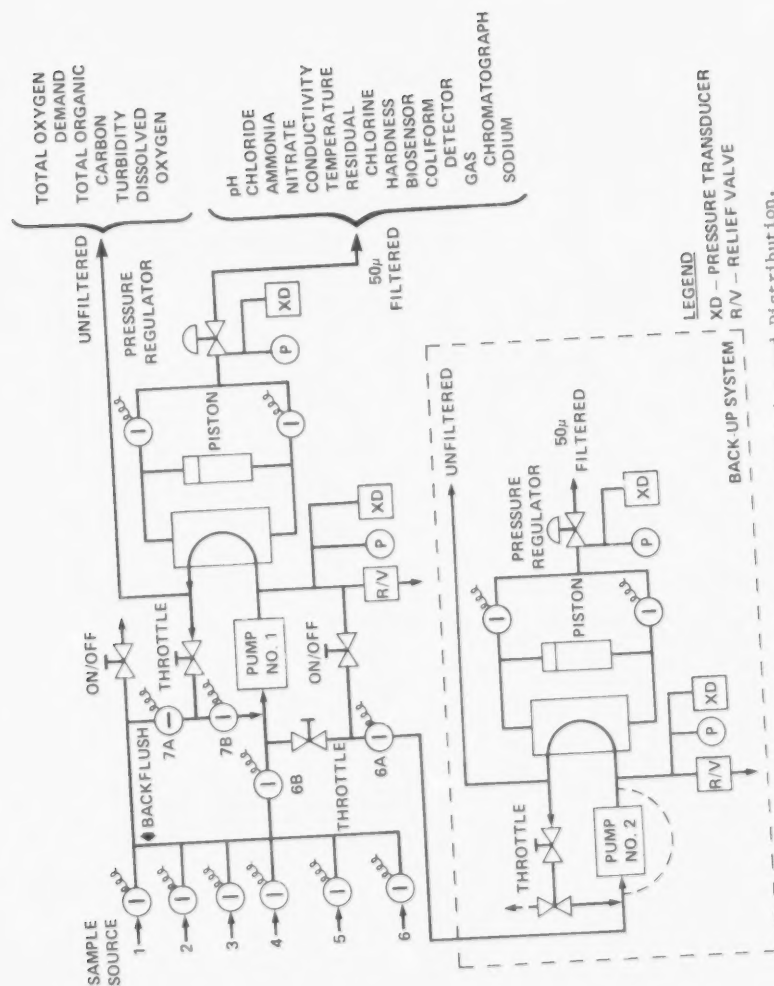


Figure 3. Sample Conditioning and Distribution.

- | | |
|-------------------------------|-------------------------------|
| 1 PRIMARY EFFLUENT | 4 CLARIFIER EFFLUENT |
| 2 RECLAMATION FAC. B EFFLUENT | 5 NOT ACTIVE |
| 3 SECONDARY EFFLUENT | 6 RECLAMATION FAC. A EFFLUENT |

6. TURBIDITY-SiO₂

3/78

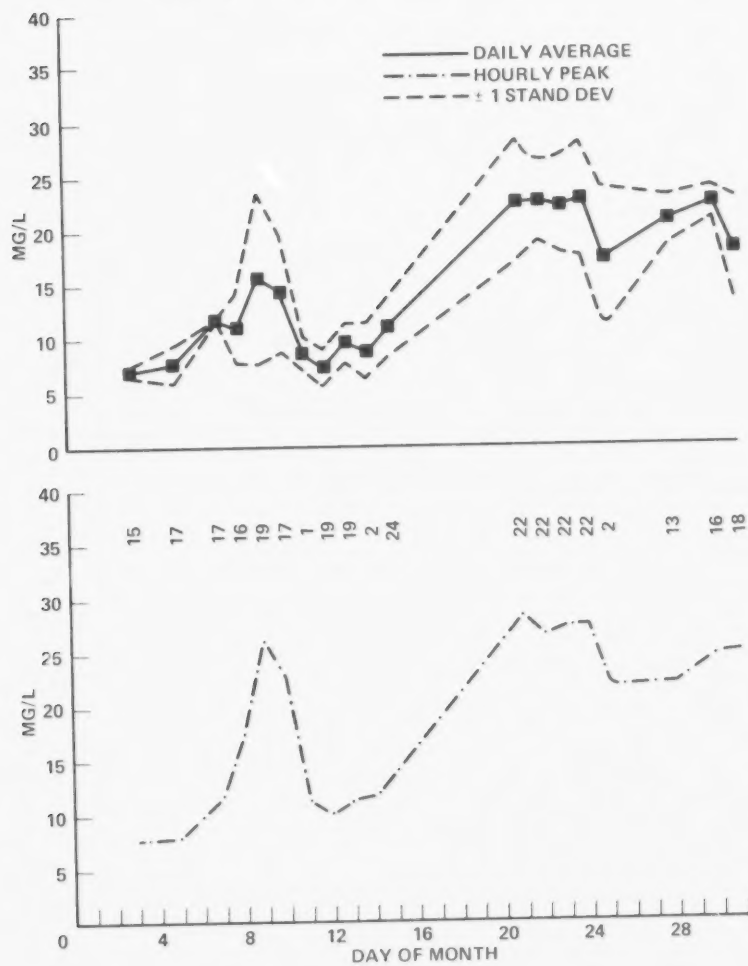


Figure 4. NASA/WMS - SCVWD Palo Alto Water Reclamation Facility.

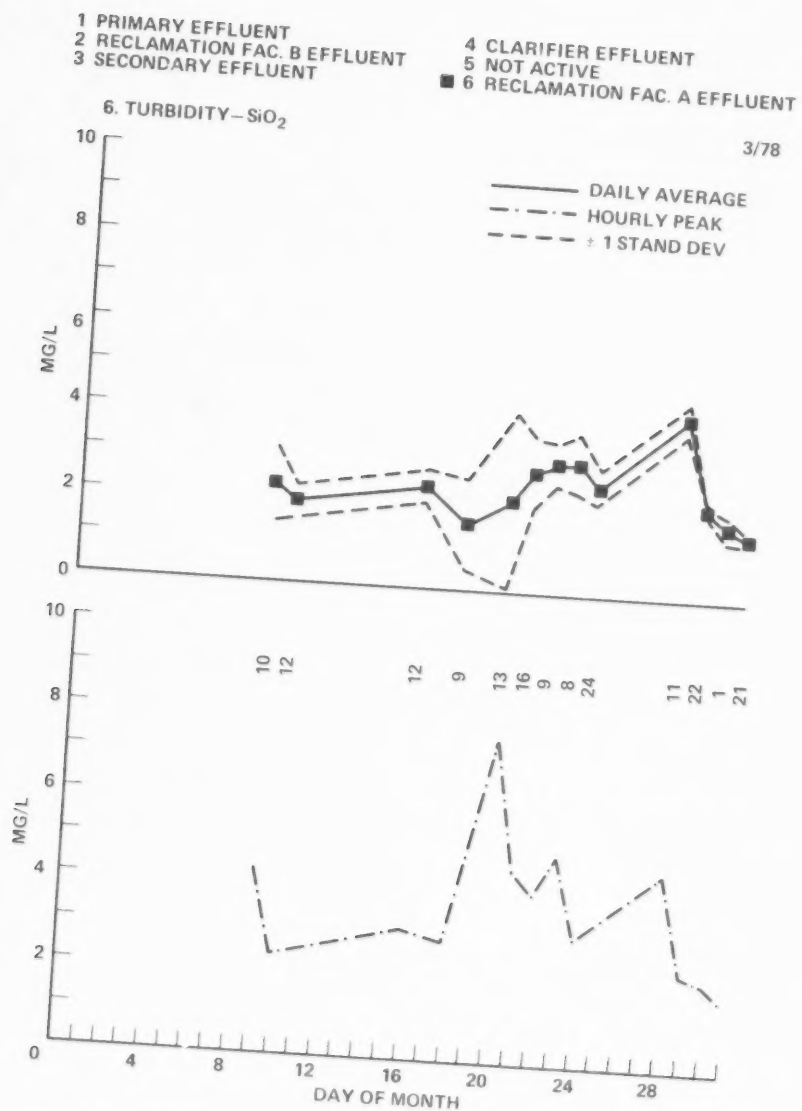


Figure 5. NASA/WMS - SCVWD Palo Alto Water Reclamation Facility.

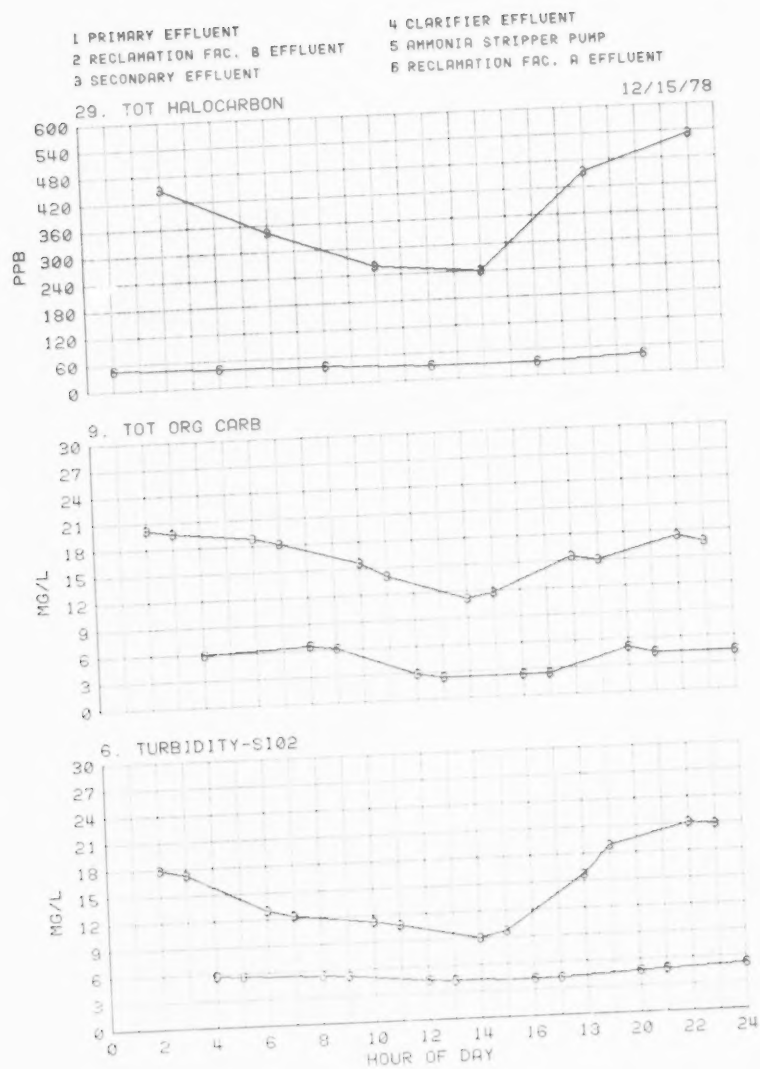


Figure 6. Correlation Between Water Quality Parameters.

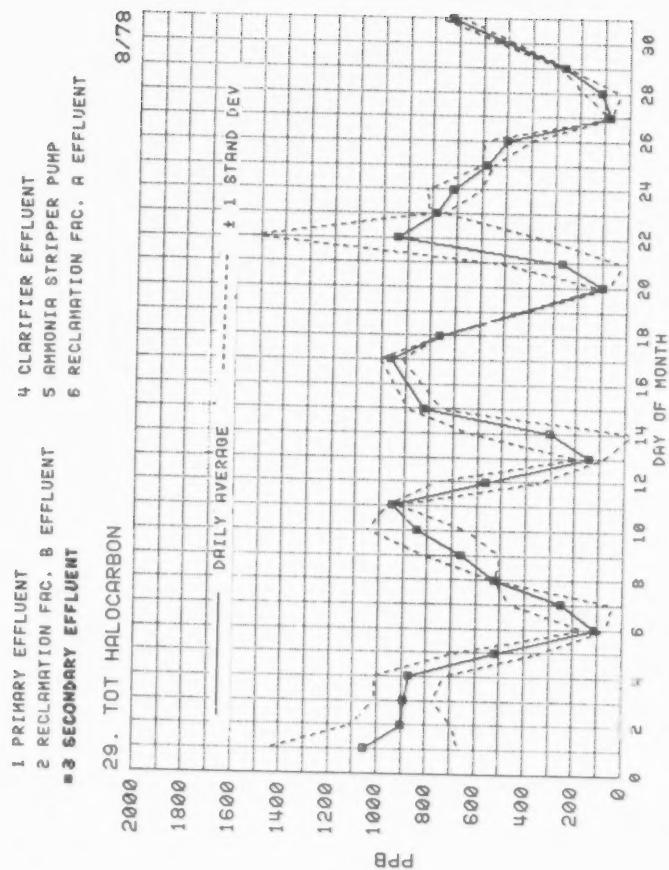


Figure 7. Total Halocarbons in Reclamation Facility Influent Water (Secondary Effluent from Palo Alto Wastewater Plant).

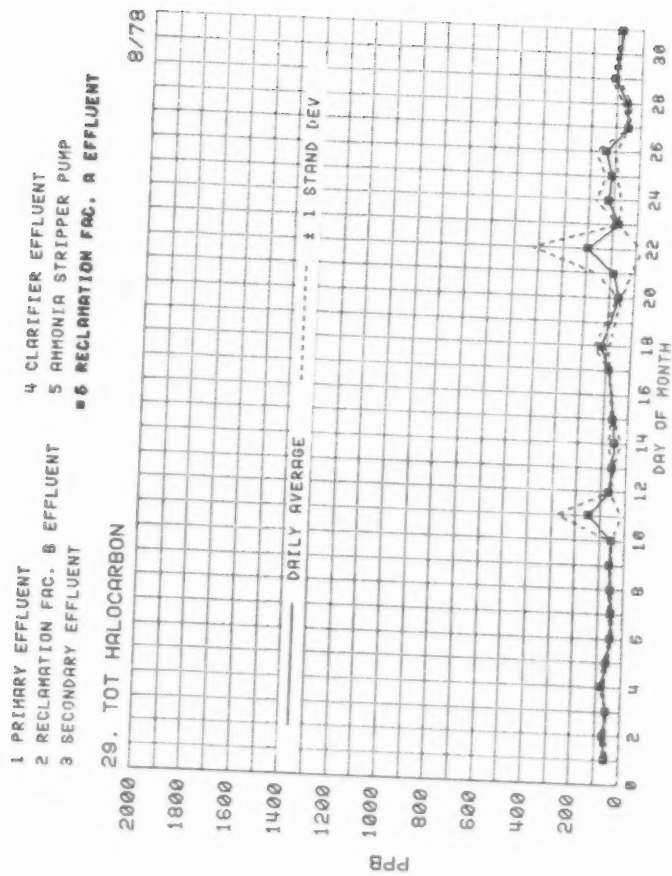


Figure 8. Total Halocarbons in Reclamation Facility Effluent.

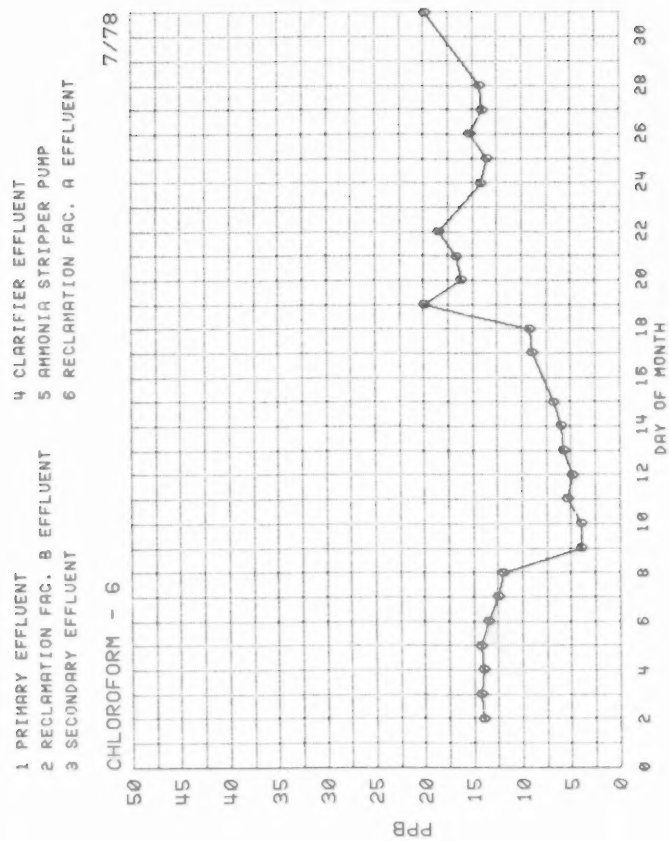


Figure 9. Chloroform in Process Water After Activated Carbon Treatment.

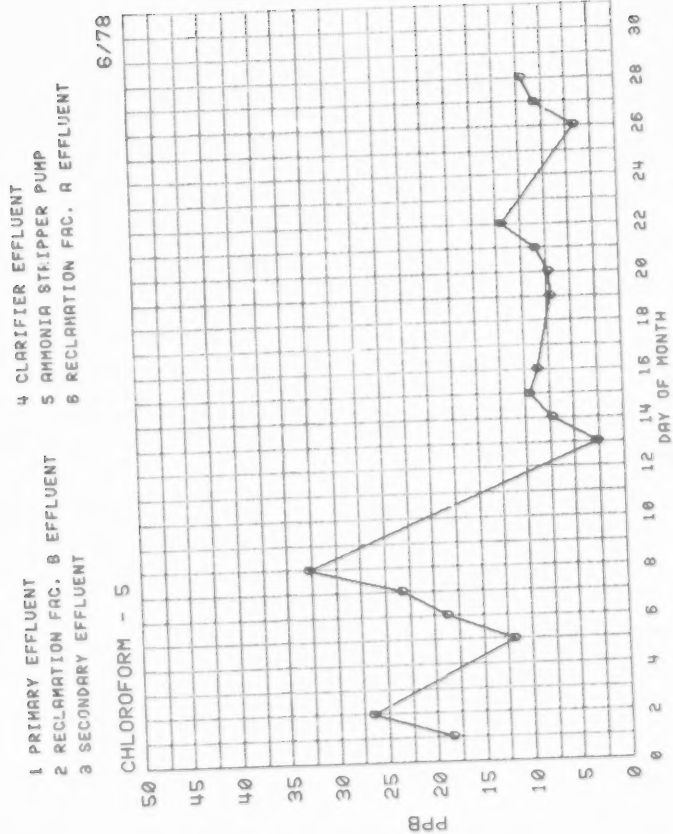


Figure 10. Chloroform in Process Water After Ammonia Stripping Treatment.

WATER REUSE MANAGEMENT USING DUAL WATER SUPPLY

By

Arun K. Deb, Ph.D., P.E.
Principal Environmental Systems Engineer
Roy F. Weston, Inc., Weston Way
West Chester, Pennsylvania 19380

INTRODUCTION

Increases in population, changes in lifestyle, and technological advances during the past decades have caused both the demand of good quality of water and the discharge of wastewater to increase. Many communities are finding it more difficult and expensive to guarantee a high quality bulk water supply for all uses. We have reached a point where it appears desirable to question the need to supply only potable quality water to meet the diversity of urban demand, since only a small fraction may require such high water quality.

In recent years, water pollution abatement requirements have resulted in the production of wastewater effluents of such quality that they may be considered sources of water for various nonpotable uses in a community.

If it is assumed that recycled water will not be employed for potable uses, a separate distribution system will be necessary for supplying recycled water for nonpotable uses. In fact, dual supply systems would be a necessary feature of water management in areas where wastewater reuse will be considered.

In a dual distribution system, two qualities of water (one potable, the other nonpotable) would be supplied through separate distribution systems. In this concept, two grades of water would be supplied to consumers according to quality and quantity requirements for various uses. A potable supply would primarily be intended for ingestion and a non-potable supply would be for purposes other than ingestion.

Water quality management in this country is now taking a new direction from technological and management points of view. In some areas of the country, there is a growing imbalance between demand and available water resources. If wastewater effluent can be reused for purposes which do not warrant high-quality water (such as urban irrigation, industrial cooling and process water, land application, and ground-water recharge), the demand on good quality natural water sources can be reduced.

The average total urban water usage for Americans today is approximately 160 gallons-per-capita/day. Nationally, the approximate categorization for the 160 gallons of per capita average daily use is as follows:

<u>Use</u>	<u>Percentage</u>
Residential	40
Commercial	15
Industrial	25
Unaccounted for	20

Vast amounts of potable quality water for industrial, commercial, and public sectors are not required. Of the 60-gpcd average interior residential usage, 40 percent is required for toilet flushing, 30 percent for bathing, 15 percent for laundering, 6 percent for dishwashing, 5 percent for drinking and cooking, and 4 percent for other miscellaneous uses.

DUAL SUPPLY STANDARDS

The quality of potable water should conform to the Primary and Secondary Drinking Water Regulations. Nonpotable water is required to meet only the microbiological portion of the primary regulations and generally the secondary regulations.

CHARACTERISTICS OF THE DUAL SYSTEM

The characteristics of the dual water system of a community depend on various factors such as the quality and quantity of available good quality natural sources and reclaimed water.

The balance between potable and nonpotable supply will vary for each community depending on its residential, industrial, commercial, and public water demands. In residences, the possibilities for potable water range from supplying all needs except toilet flushing and exterior uses, to providing only potable drinking and cooking water, and using nonpotable water for the remaining household uses. If reclaimed water is used for nonpotable uses, it can be used for toilet flushing, urban irrigation, land application, industrial cooling and process water, air conditioning, groundwater recharge, and other low-grade water uses. The breakdown of potable versus nonpotable water for each demand combined with the total levels of water usage will determine the character of the dual water system.

Objectives of a long-term urban total water management plan should include:

- Reduction or elimination of the risk of health hazard resulting from continuous ingestion of low levels of toxic contaminants over a period of years.
- Conservation of good quality water.
- Reduction of urban wastewater effluent and conservation of good-quality water by reusing wastewater.
- Development of an overall efficient and economical long-term management plan to meet various urban needs of water in terms of quality and quantity.

WATER REUSE MANAGEMENT MODEL

Any wastewater reuse management planning for a community should be done in conjunction with the overall long-term water management planning of the community. In order to achieve an efficient and economical long-term water management plan including reuse, it would be required to consider more than one water source and more than one quality of water supply. This concept may produce a large number of alternative plans for supplying water in order to meet various demand needs of a city or a region. Some of these possibilities can readily be dismissed on the

basis of engineering judgment and local conditions. However, a substantial number of reuse alternatives will require full analysis and evaluation before selection of a final plan.

With a grant from the National Science Foundation, Weston has developed a WATER MANAGEMENT (WATMAN) model (1). The WATMAN model provides technical and economical evaluation of various alternatives of long-term planning and management of the urban water reuse system, and helps urban water and wastewater managers to find the most economical and implementable plan for their cities.

A general conceptual diagram of the WATMAN urban water system model is shown in Figure 1. A model of the quantity and quality aspects of an urban water network would incorporate the following three basic components:

1. Water sources, including recycled water.
2. Water demand by quality.
3. Water treatment (including treatment of wastewater after secondary treatment) and distribution (including more than one distribution system).

The model outline shown in Figure 1 may have numerous options, depending on: the quality and quantity of sources; potable and nonpotable water demand; and institutional and political considerations. This model provides the user with a tool to analyze various integrated water supply system alternatives, including reclaimed water distribution, depending on the composition of sources and demands. Water reuse would be an essential consideration in the future in developing any long-term water management plan for a city, industry, or a region. The WATMAN model will be particularly useful for analyzing water reuse management plans. At present, this model can only be used as a planning tool, rather than a design tool.

In analyzing a dual supply system, based on the sources and treatment requirements, the potable and nonpotable system may be completely or partially separate. In the case of water reuse, a nonpotable system will be completely separate. Total present worth of collection, transmission, treatment, and distribution of a water system of any configuration for conventional or dual supply can be calculated using this model.

In formulating the mathematical model, parameters such as potable to total flow ratio, interest rate, annual capital and operation and maintenance (O&M) cost increase rates, and cost function are considered as variables.

The systems model that has been developed in this study is a powerful tool for analyzing the various alternatives of long-term total urban water management. The model is also very flexible and can accommodate any configuration of water supply system. It can handle: a conventional system, and a dual system with more than one source of water (including reuse) and one supplying more than one grade of water to various demand centers.

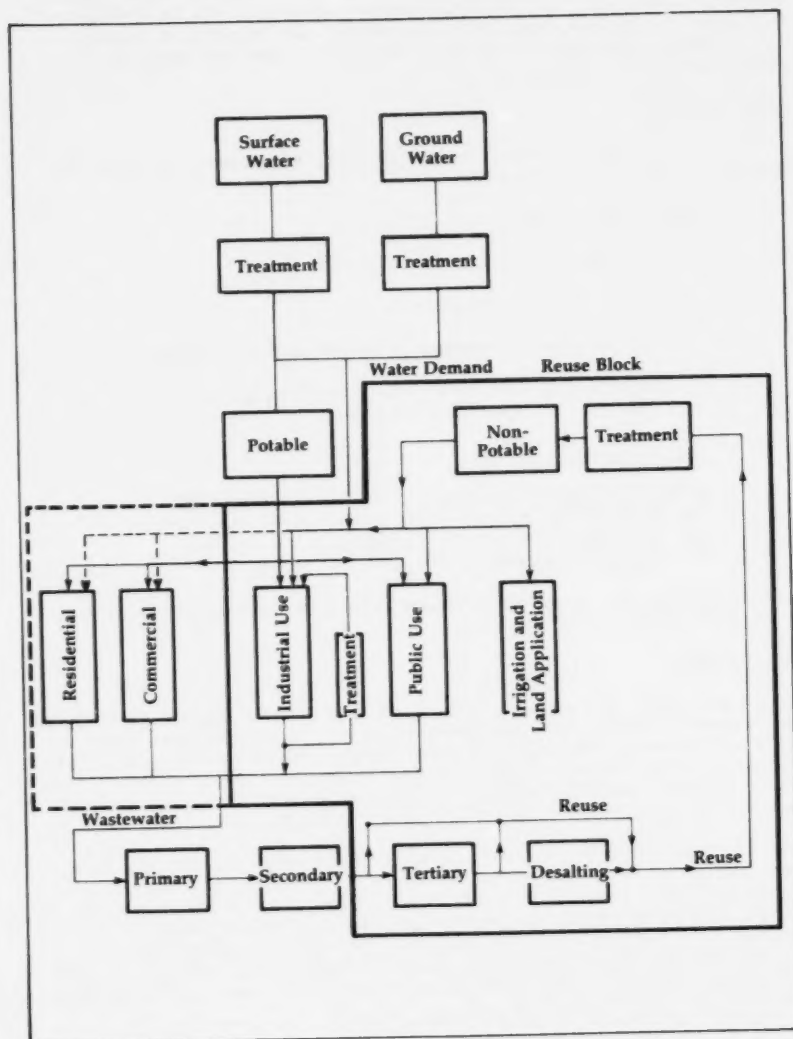


FIGURE 1
URBAN WATER REUSE MANAGEMENT CONCEPT

The model has also been designed to accommodate regional management of water, with up to 10 cities in the region. In a regional water supply management analysis, one limited good-quality source may be considered to supply all potable demands of the region, and nonpotable demands can be drawn from local unprotected sources or from effluents of sewage treatment plants.

WATER SOURCES

In this study, the following sources of water have been considered for potable and nonpotable supplies:

1. For Potable Supply:
 - a. Good quality protected groundwater.
 - b. Protected upland reservoir.
 - c. Unprotected groundwater after extensive treatment.
 - d. Unprotected surface sources (polluted streams, rivers or lakes) after extensive treatment.
2. For Nonpotable Supply:
 - a. Unprotected groundwater after normal treatment.
 - b. Unprotected surface water after normal treatment.
 - c. Secondary effluent for reuse.
 - d. Advanced wastewater effluent for reuse.

For nonpotable use, secondary effluent should be followed at least by filtration and chlorination.

APPLICATIONS

Using this model, any water supply system new or existing can be analyzed for long-term planning and management. Depending on the sources and demand requirements, the user must identify the physical configuration of various alternative water supply systems to be analyzed, proceeding from source to distribution. Several alternative systems should be analyzed in order to determine a cost-effective implementable solution.

Many hypothetical water systems under new and existing conditions have been studied using this computer model for conventional and dual water supply systems. Sensitivities of potable/total flow ratios in a dual system, inflation rates, and population densities on the costs of conventional and dual systems are studied. The computer output includes O&M and salvage values, and total present worth of each system. Common examples of reuse of wastewater for conservation of good quality sources are shown in Figures 2 and 3.

In an industrial city, where a large portion of the municipal water demand is for industrial needs, a dual supply can be effectively used with one nonpotable supply for industries and a potable supply for other uses. The industrial nonpotable supply can be derived from the same source with only existing treatment, or from reclaimed municipal wastewater with further treatment. Figure 4 shows a simple schematic of alternatives for an existing system on a polluted river which has been analyzed with this model.

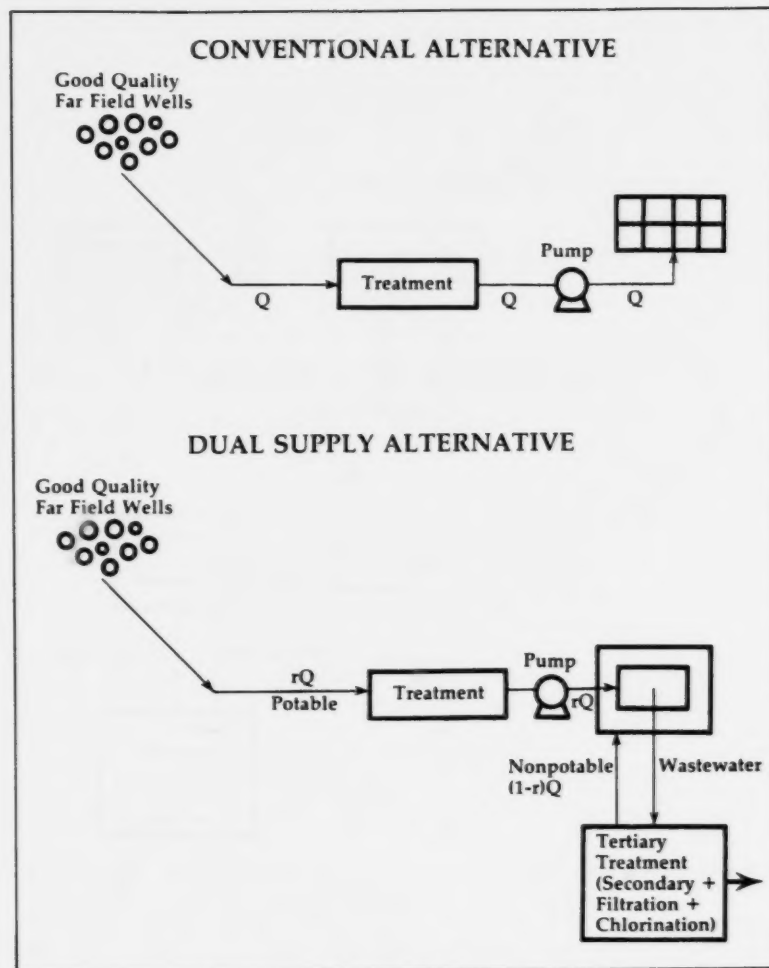


FIGURE 2
SYSTEM 1-2
CITY: 100,000 POPULATION

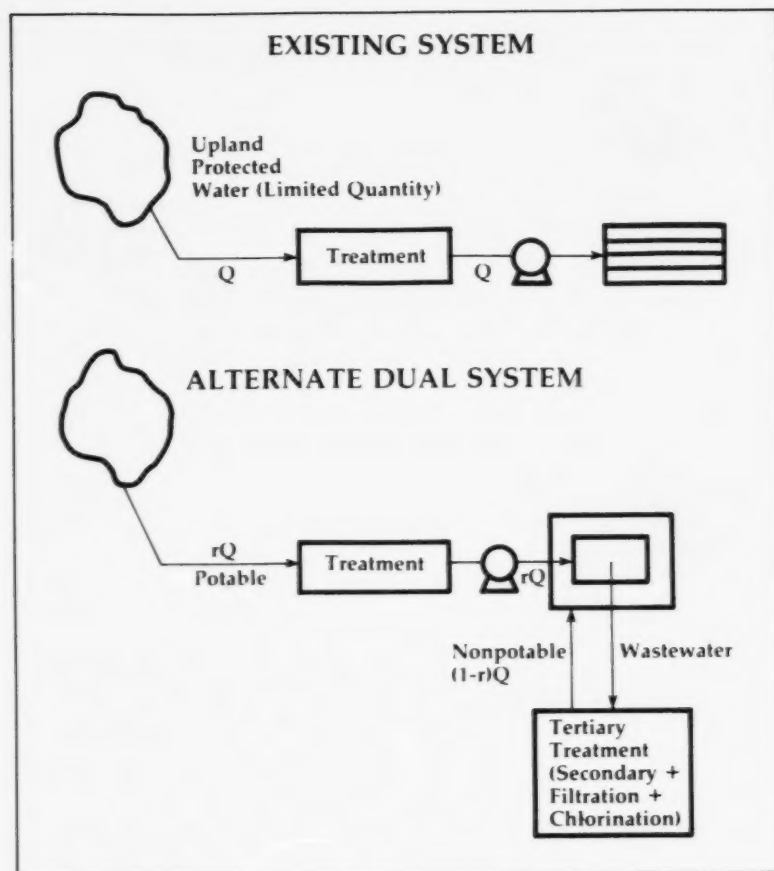


FIGURE 3

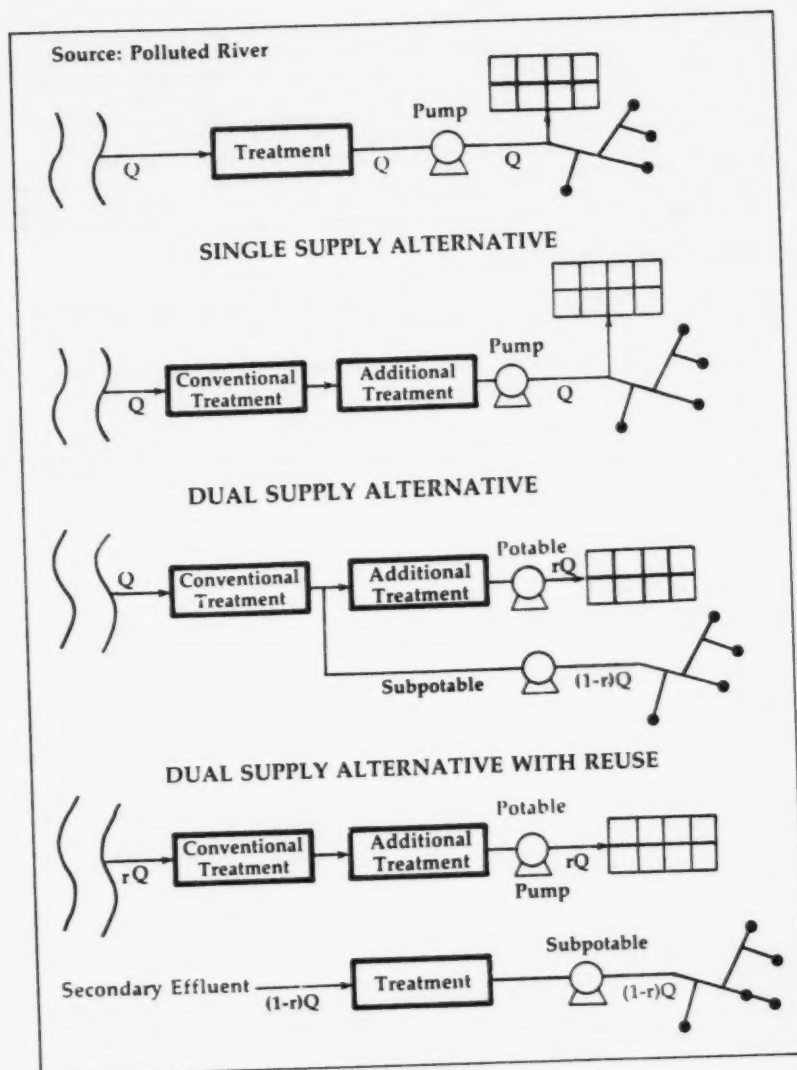


FIGURE 4
EXISTING SYSTEM
FOR INDUSTRIAL USES

Regional Applications

In regional management of water, good-quality-limited sources can be conserved and used for potable supply only through a regional water treatment and distribution system. The water for nonpotable uses can be obtained from reclaimed water (Figure 5). In a region where a limited quantity of protected water is available, existing communities obtaining their water supply from nearby polluted sources will have a choice. They will have the option of having additional treatment for removing trace chemicals, or using a limited protected source for potable supply to all communities in the region using a dual distribution system. The system shown in Figure 5 has been analyzed using the model (WATMAN).

Tampa Test Case Reuse Study

As a part of this study, the City of Tampa water system and possible future use of reclaimed AWT effluent have been analyzed.

The quality of the well water in Tampa is good, but Hillsborough River water quality varies considerably. With the proposed safe drinking water act regulation, Tampa would have to treat the Hillsborough River water beyond softening, color removal, filtration, and chlorination. For this test application, it is assumed that the Tampa Water Department will have to modify its treatment by adding activated carbon to reduce the level of total trihalomethanes below the 100 $\mu\text{g/L}$ proposed maximum contaminant level.

The City of Tampa will need about 122 mgd to meet its maximum daily demand in the year 2000. This will require an additional 22 mgd to the existing supply. Figure 6 indicates schematics of the existing system and proposed future conventional alternative with activated carbon added to the Hillsborough River Water Treatment Plant (HRWTP).

Figure 7 indicates the schematic for the water reuse dual supply alternative being analyzed with the WATMAN model. In the dual alternative, the 40 mgd from the Morris Bridge well field will supply all residential needs except toilet flushing and exterior usage. The Hillsborough River flow will be augmented by Tampa City AWT effluent.

Cost data of various unit processes were collected from the region (the City of Tampa and the City of St. Petersburg) and analyzed. Cost function coefficients and exponents valid for the region have been derived for use in the computer model WATMAN.

In simulating each of the systems, the capital and O&M costs expected to incur during the planning period (1978-2000) to supply water to meet peak day demand requirements are considered. In this study, it has been assumed that the potable water quality as a minimum should meet the primary and secondary drinking water regulations. Nonpotable supply should meet secondary drinking water regulations and bacteriological quality requirements of primary water regulations.

Since good quality protected groundwater is available (Morris Bridge Well Field) to protect public health from long-term ingestion of trace chemicals, protected groundwater has been conserved only for potable uses for the City of Tampa.

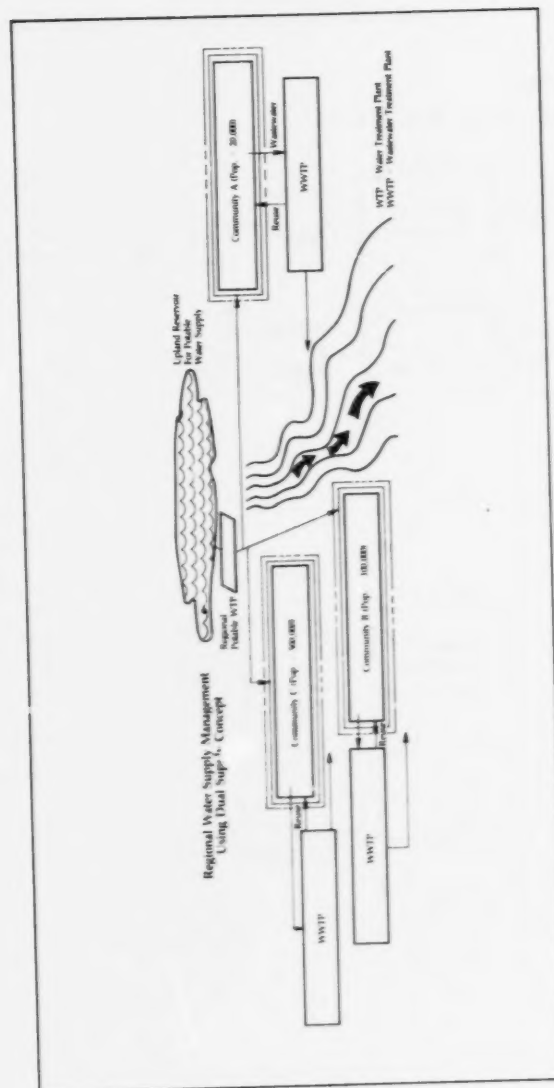
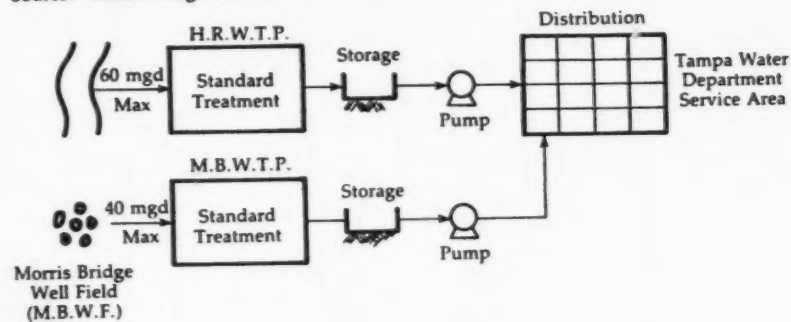


FIGURE 5
REGIONAL WATER SUPPLY
MANAGEMENT USING
DUAL SUPPLY CONCEPT

Existing System

Source: Hillsborough River (H.R.)



Conventional Alternative

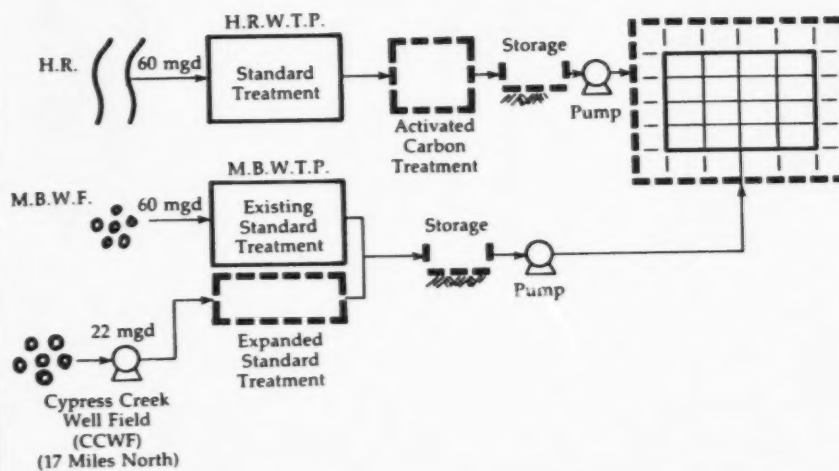


FIGURE 6
EXISTING SYSTEM AND CONVENTIONAL
ALTERNATIVE FOR THE
TAMPA, FLORIDA TEST CASE

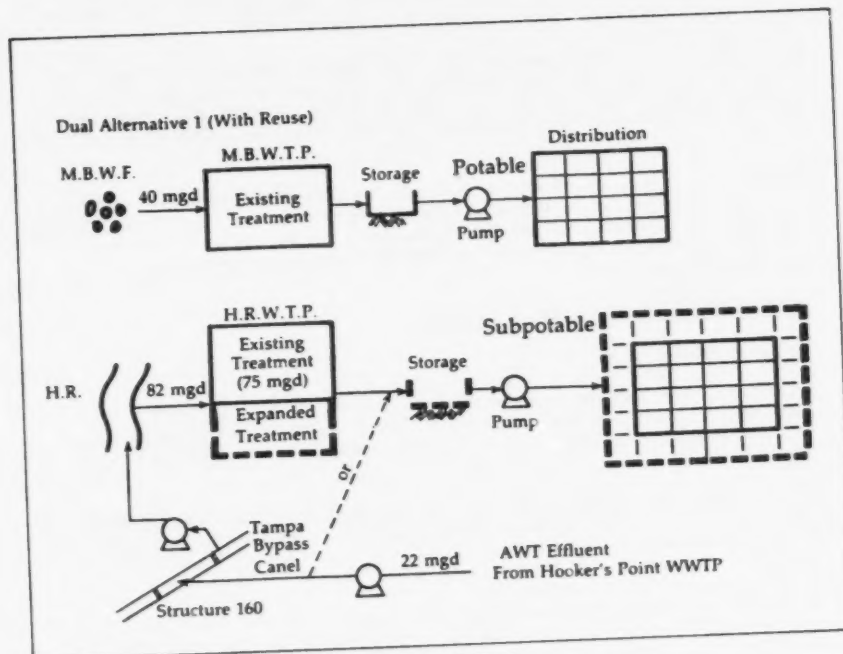


FIGURE 7
DUAL WATER SUPPLY ALTERNATIVES
TAMPA, FLORIDA TEST CASE

The results of the systems expressed in terms of the present worth cost ratios (dual/conventional) with flow ratios (potable/total) for the Dual Alternative have been plotted in Figures 8 and 9.

Since the O&M cost of granular activated carbon treatment is high, the O&M inflation rate has been found to be very sensitive to the total present worth of the conventional system which includes GAC treatment. All the systems have been analyzed with O&M inflation rates of 6 and 10 percents.

Therefore, it appears from the analysis that the present worth cost of the Dual Alternative is comparable with the present worth cost of the conventional system. If the O&M inflation rate is about 10 percent, the dual system is expected to be cheaper than the conventional system for the City of Tampa.

WATER REUSE REGULATION AND GUIDELINES

General Regulations

In order to compile nationwide regulations and guidelines regarding dual water supply and water reuse, a survey has been conducted among all the State Health Departments. Thirty-seven states replied to our request.

Four states (California, Colorado, Kansas, and Utah) have regulations regarding treatment of wastewater and quality of effluent to be used for urban landscape irrigation. All the states as a minimum require secondary treatment and disinfection of effluent for landscape irrigation. However, the state of Utah requires AWT effluent for irrigation of public parks and lawns with BOD and suspended solids less than 5 mg/L and total coliform of 1/100 ml. The state of Kansas does not have any requirement for quality of effluent to be used for urban landscape irrigation, but does require secondary treatment of wastewater followed by filtration and disinfection.

Virus standards have not been adopted. Where the reclaimed waters are accessible to the public, there is a need to establish virus limits and virus monitoring. However, technology is not now adequate, and standards have not yet been developed.

The coliform limit, similar to those for drinking water (1/100 ml), is probably adequate.

Monitoring and Reliability

The following quality and monitoring measures are suggested for the distribution of reclaimed wastewater for nonpotable uses:

- Secondary maximum contaminant levels should be met, with some relaxation of limits for dissolved solids as appropriate.
- Routine viral and bacterial monitoring should be done. Coliform should not be greater than 1/100 ml (primary drinking water standard) and virus standards will need to be established.
- Turbidity before post chlorination should not be more than 1 unit (primary drinking water standard).

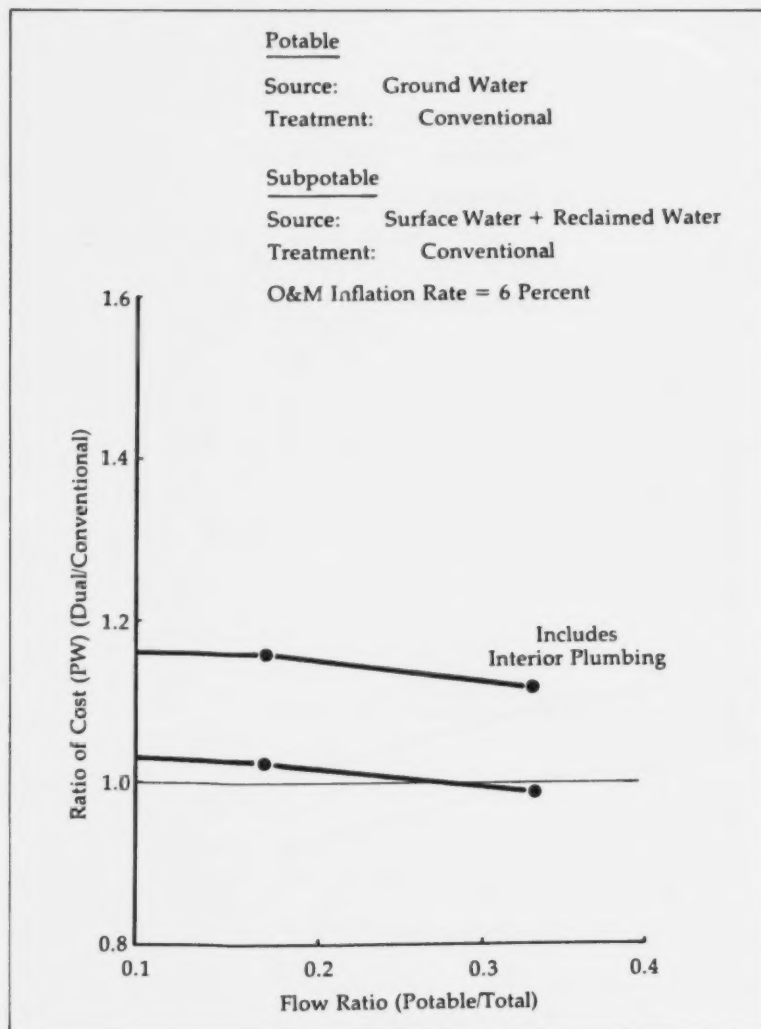


FIGURE 8
DUAL ALTERNATIVE 1
TAMPA TEST CASE

Potable

Source: Ground Water
Treatment: Conventional

Subpotable

Source: Surface Water + Reclaimed Water
Treatment: Conventional

O&M Inflation Rate = 10 Percent

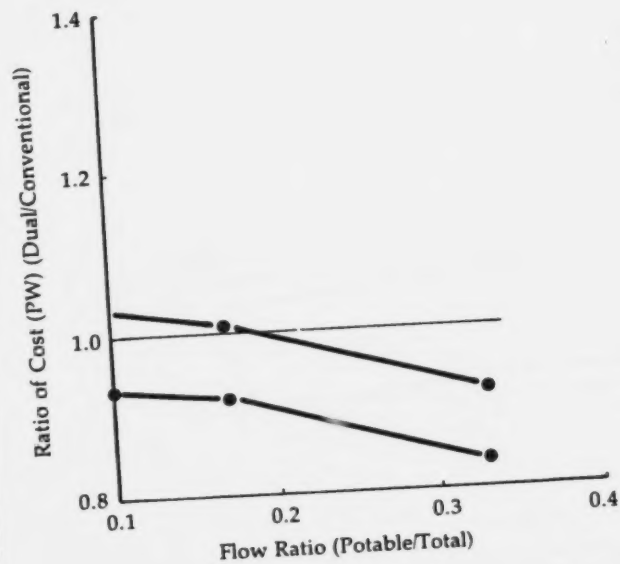


FIGURE 9
DUAL ALTERNATIVE 1
TAMPA TEST CASE

- Plant effluent free chlorine should be more than 3.0 mg/L, and system residual should be more than 0.5 mg/L.

It is expected that reliably operated secondary treatment followed by polishing and chlorination would be adequate to meet these criteria.

To increase the reliability of the performance of the system, back-up units for each unit process are desirable. Some of the critical unit operations (power supply, pumps, disinfection, storage for diverted reclaimed water, etc.) must have back-up units in order to supply the reclaimed water of reliable quality.

CONCLUSIONS

The systems model developed in this study has been tested with many hypothetical test cases and two actual test cases. From these applications, it has been demonstrated that the model is an efficient, practical, and easy-to-use tool for engineers and planners for technical and economical evaluation of various conventional, dual supply, and water reuse alternatives for long-term community water supply management. This study showed that it is economically feasible to consider dual water supply and water reuse as viable alternatives for conventional management under many circumstances.

In the future, the cost of supplying good quality potable water will be high, and, with availability of good quality of wastewater effluent in many cases, there will be an economic incentive to reuse wastewater for nonpotable uses. There are already many cases where water is reclaimed and used for bulk uses. In the future, many water reuse systems would be extended in residential and commercial establishments for toilet flushing and other nonpotable uses. However, proper judgment and wisdom must be used to increase the reliability of reuse systems and to eliminate health risks.

REFERENCE

1. Deb, A.K., "Multiple Water Supply Approach for Urban Water Management," Final Report Submitted to the National Science Foundation, 15 November 1978.

ACKNOWLEDGMENT

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SYSTEMS ANALYSIS TECHNIQUES APPLIED TO WATER REUSE PLANNING

Myron S. Rosenberg, Ph.D.
Associate Engineer, Water Resources Division
Camp Dresser & McKee, Inc.

Harvey O. Banks
President, Water Resources Division
Camp Dresser & McKee, Inc.

INTRODUCTION

In order for wastewater reuse to realize its true potential, it must be considered as part of a regional water resources management plan...Digital computer models and linear programming techniques have been applied successfully as aids to resource planning, but their use in the field of wastewater reuse is in its infancy. (Reference 13; emphasis added).

Analysis of systems that include reuse is in its "infancy" because reuse itself-- as one component of a larger water/wastewater system-- is only now proving to be a generally viable technological and economic option. The ability to exploit a water resource system's reuse potential can lessen the demand for capacity-expansion of traditional water supply sources. Incorporating technologically feasible reuse possibilities into water resource systems planning, however, does not require wholly new system's analysis* techniques; many of the tools which are in use today can address reuse.

From a systems point of view, reuse schemes are conceptually no different from other water/wastewater system components: costs and benefits (neither being limited to monetary measures) can be associated with the facilities necessary to obtain wastewater, treat it to suitable quality, and deliver it for reuse, the same as if the original source was a reservoir or well-field. However, matching treatment levels to user quality requirements, the possible need for separate distribution systems, and the many social/political (attitudinal) obstacles associated with implementing reuse options, make modeling systems that include reuse somewhat more complex than modeling conventional systems. Given these complexities, it is necessary for planner and administrator alike to thoroughly understand the advantages and limitations of the analysis tools which are likely to be increasingly applied to systems which include reuse possibilities. In particular, it is important to appreciate the planning issues which can be addressed with systems analysis, the nature of the information such analyses can produce, and any inherent assumptions or simplifications which should be kept in mind while interpreting analytical results.

*Throughout this discussion, the term "system analysis" will generally be used to connote both modeling techniques and the broader public expenditure theory associated with the development and use of such models.

WATER RESOURCE SYSTEMS PLANNING

The use of systems analysis for planning complex, interrelated water resource projects was first thoroughly explored during the "Harvard Water Program" which was conducted between 1955-1960. The principal methodological results of this work were published as Design of Water Resource Systems in 1962 (Reference 6). Now, 17 years after its publication, many of the ideas first conceived by the participants in the Harvard Water Program are in widespread use, and have been revised and improved over the years. Current planning techniques rely heavily on mathematical modeling analyses. Many general-purpose optimization and simulation models have been developed by various government, academic and private planning practitioners.

Eight interactive steps in the water resources planning process are shown in Figure 1 (Reference 10). Today, what are labeled Steps 3,4,5,6 and 7 in the figure are most appropriately tackled with the aid of system-wide analysis (Reference 7). Figure 2 illustrates why this is so: In theory, systems of individual development projects (which could include reuse schemes) help serve purposes such as domestic water supply, irrigation, recreation, etc. These purposes, in turn, should meet national/regional objectives such as economic efficiency, employment environmental quality, etc. These objectives can be quantified, and the performance of potential development systems measured in these terms. Thus, the metrics of system performance are used to guide the system design. This is, of course, an iterative process.

In this planning/analysis context, water reuse is treated as any other system component. Thus, systems analysis techniques can help to answer some fundamental questions regarding reuse as part of the total system. These include such considerations as the scale of reuse facilities that may be required; the degree of quality and reliability necessary to meet various demands; the optimal implementation schedule for reuse system components, etc. To score the point, in Decision to Reuse Water, Baumann and Dworkin stated: "To be effective... model[s] must be able to simulate the [system] process and test [system] assumptions. We believe that unless such an evaluation is made, reuse will often be [judged] an expensive additional method of supply rather than a flexible efficient component of the water supply system." (Reference 1).

There are two major categories of models that can be incorporated into the water resource system planning process. They are: (1) optimization models, and (2) simulation models. Taking a broad view of the planning process (as illustrated in Figure 1), experience with complex river basin planning problems has shown that both model-types are required to yield all necessary information (Reference 3). Inherent features of optimization models make them useful for "screening" purposes, but unrealistic for detailed evaluation. Simulation tools are necessary to examine and evaluate system options which have been identified as "attractive" by screening.

OPTIMIZATION MODELS

Optimization models are based on mathematical programming techniques. A mathematical function is formulated to express some quantity one wishes to optimize (maximize or minimize) subject to a set of constraints which are also expressed mathematically. This function is known as the "objective function" and, as the name implies, it expresses the objective(s) of interest (e.g., minimization of cost, maximization or net benefits, maximization of environmental quality concerns, etc.).

A linear optimization model which has often been adapted to water resource system planning problems is known as the "transportation problem." Figure 3 illustrates the traditional form of the transportation problem as it might be applied to water supply problems. The objective is to minimize cost-- the x_{ij} 's can be viewed here as representing the amount of water transported from source i to user j ; the c_{ij} 's represent the unit cost of transporting (treating to suitable quality, pumping, etc.) water from source i to user j . The constraints simply state that supply (S_i) cannot be exceeded at any source, nor can demand (D_j) remain unsatisfied for any user.

The transportation problem was adopted and used by Bishop and Hendricks (Reference 2) to examine the possibility for incorporating reuse in the Salt Lake City water supply system. This work has become somewhat of a classic; two illustrative results are shown here as Figure 4. These results are useful in exploring both the advantages and disadvantages of optimization models and, hence, why they are useful as "screening" techniques.

ADVANTAGES

What is perhaps the major advantage of optimization modeling is directly evident from Figure 4. That is, it is possible to alter "demands" and examine whether and how the "optimal" supply allocations are effected. The ease with which it is possible to examine various planning scenarios make optimization models quite useful in getting a feel for what sorts of systems seem likely under various conditions. For instance, in the 1965 optimal allocations, desalting, blending, and tertiary treatment are not included in the system. Thus, if one wished to simulate the system in detail (see below), it would not be necessary to include these components.

The flexibility associated with optimization models stems from two inherent facts. First, once such a model is formulated and various coefficients determined, it is relatively inexpensive to make alternative computational runs. Second, the mathematical techniques associated with optimization models are easily analyzed for what is known as "sensitivity" information. That is, the analyst is quickly able to deduce important decision-making information such as which constraints are binding (and what the "shadow price" is associated with relaxing those constraints); whether minor changes in any cost coefficients would alter the optimal solution, etc. A skilled analyst can glean a great deal of policy/planning information from the use of an optimization model. Most significant, of course, is a firm understanding of which system components (at what scale) are likely to be incorporated into the final system design. This "screening" process reduces the number of possible system configurations which must be subjected to more detailed analysis with simulation.

DISADVANTAGES

Although optimization models help to screen the components likely to be included in complex systems, the nature of the assumptions necessary for their formulation indicate many weaknesses. This is why simulation models are also necessary to execute a thorough water resource system planning exercise. It must be understood, however, that there are many degrees of sophistication possible in the formulation of optimization models. The linear transportation problem is among the simplest. In general, though, the less complex the formulation, the less costly the model

formulation work and computation time, but the more unrealistic the system assumptions. Tradeoffs are involved.

To illustrate some of these tradeoffs, consider the transportation problem as illustrated in Figure 3. Inherently, by assigning a single cost coefficient (c_{ij}) to a unit of water transferred from source i to user j , there are assumptions of linearity. However, what about start-up costs? What about economies of scale? These real-world problems are assumed away in this simple set up. Moreover, this formulation merely assumes "generic categories of sources and users. That is, all industrial users, agricultural users, etc., are lumped together: the sources too are specified by group. It would be possible, of course, to be site-specific and formulate the transportation problem to consider individual users and sources and thereby establish specific cost coefficients that would more accurately reflect necessary quality improvements and pumping/piping costs. The greater number of "decision variables" in this formulation would also help reduce assumptions associated with economies of scale, since smaller ranges of values would be involved and over which the cost coefficients must be representative. However, formulation and computation costs greatly increase as a function of the number of decision variables involved.

As water resource planning optimization models which include reuse possibilities evolve and as operations research efforts continue to develop new and better solution algorithms, it is possible that more and more of these inherent weaknesses will be eliminated. For example, techniques for handling non-linear objectives and constraints already exist and are continually being improved (References 8 and 12). In fact, in the February 1979 Water Resources Research, a paper is presented which formulates a non-linear optimization model specifically designed for reuse planning (Reference).

Present and future improvements in techniques, however, will never be able to obviate the most glaring weakness of optimization models. They rely on time-invariant supply and demand specifications. Thus, while optimization models help to perform long-range forecasting and "screen" likely and attractive system components, they do not and cannot take account of temporal variability in supply or in demand. Since there may be daily, monthly, seasonal, and annual variance in both, when considering reuse especially, it is necessary to use simulation techniques to complete the water resource system planning process.

SIMULATION MODELS

As the name implies, simulation models mathematically reproduce (simulate) the time-varying behavior of a water resource system. The temporal resolution at which simulation is performed and the degree of detail incorporated in the model should reflect the nature of the planning problem at hand. For large-scale river basin simulation, a monthly trace of streamflow is often used as the input and operational behavior is examined with a monthly time step as well.

For reuse planning-- where even the daily fluctuations in supply and/or demand may be significant-- it may be necessary to "model" at a more refined time scale than has been used in the past for most river basin planning efforts. Also, because the area-of-interest associated with reuse potential may be relatively small, the system boundaries may not necessarily coincide with river basin boundaries. Nonetheless, in their conceptual approach, simulation models now being utilized for river basin planning purposes appear amenable to adaptation for reuse. As a result, much of the discussion to follow is based on a simulation model called RIVSIM. This model was originally developed by students and faculty in the Department of Civil Engineering at the Massachusetts Institute of Technology and was referred to as MITSIM (Reference 11). RIVSIM has better and more detailed groundwater capabilities and was adapted from MITSIM by personnel in the Water Resources Division of the environmental consulting firm of Camp Dresser & McKee, Inc. (CDM).

RIVSIM provides significant input in the operational analysis and economic evaluation of development options for a water resources system. The model (or its MIT-developed predecessor) has been employed in several large-scale applications both in the United States and abroad. RIVSIM is user oriented and general purpose. Specifically, the model assesses the future operational and economic consequences of alternative water resource development systems that are being considered for implementation.

One aspect of the above description is particularly important. MIT's experience in several river basin planning projects led the original model developers to create a single simulation tool that would address both operational and economic performance. As will be shown, this is important and simulation analyses that includes reuse should address both issues. Before demonstrating this, however, it is necessary to generally discuss the RIVSIM model and postulate how reuse could be handled.

The RIVSIM model generates operational and economic performance information by replicating the actual behavior of a system subjected to a long-term trace of varying hydrologic input (streamflow). The different activities which take place throughout the system over time are simulated in a straightforward manner. The components of the system under study are conceptualized as a series of nodes connected to adjacent upstream and downstream nodes by hydraulic connections referred to as arcs. The user supplies input for the physical, hydrologic, and operating features associated with each node. With this information, necessary calculations can be performed to obtain an accounting of the flow at each node at each (monthly) time step of the simulation period. The operation of nodes (system components) is simulated at a "medium" level of detail compatible with the intended planning purpose for which the model was designed.

Figure 5 illustrates a hypothetical river basin development plan. Figure 6 illustrates how this hypothetical layout would be schematized for RIVSIM. Evidently, the current version of RIVSIM is not designed to address direct water reuse possibilities. Municipal and industrial use (M&I) nodes are simply modeled as points of water withdrawal (and return flow) as shown in Figure 7. A set of monthly targets for M & I withdrawal is specified as input. The target demand is supplied each month (if available) according to preset operating policies that are formulated to consider environmental and other constraints such as maintaining certain minimum flows in the stream channels. Details of the M & I use and delivery system are not considered.

Figure 8, however, illustrates how an M & I system could conceivably be modeled to interact with the overall RIVSIM setup. Essentially, this approach would involve establishing new node-types and incorporating the appropriate quality and operational issues into the model. To be compatible with the overall detail of river basin analysis, it would be necessary to consider generic rather than site-specific user groups.

Simulating reuse at a time and spatial scale appropriate to river basin planning, however, is probably inappropriate. Instead, it is more likely that reuse planning will necessitate that separate simulations be performed for each individual M & I area. This is really a question of the system boundaries one "draws" when performing a planning exercise. However, a detailed (short time step, site specific) simulation tool that addresses reuse could be established along the same lines as RIVSIM.

There is little danger of the reuse system complexity being beyond the capabilities of simulation. Many different alternatives for the river basin system shown in Figure 9 were modeled using RIVSIM. The model was able to perform and keep track of all required calculations with performance reliability and economic evaluations generated for each of the alternative systems and their components. It is with respect to these built-in operational/economic analyses that the most useful information is obtained from the system simulation.

PERFORMANCE ANALYSIS

RIVSIM is designed to explicitly examine how well various system components "perform" over the length of the simulation period. The model is provided with monthly "target" levels of water requirements as input associated with those nodes which use water. Operating rules are established for supply allocations when and if demand cannot be satisfied in any particular time increment of the simulation. In the present version of RIVSIM, M & I nodes are among the most simply modeled. Power generation and irrigation nodes are modeled in much greater detail (Reference 5). Hence, although RIVSIM's M & I performance analysis output will be illustrated here (see Figure 10), it must be remembered that more complex analysis and output is possible. Thus, in simulation involving reuse, it would not be difficult to include analysis of water quality performance in relation to water quality requirements associated with various user nodes.

As can be seen from Figure 10, RIVSIM performance output is designed to display information concerning supply "reliability." This information -- when examined for many nodes in the system -- can help the analyst determine where and how system adjustments might improve or worsen reliability at different locations in the river basin. However, this information is most useful when combined with the economic evaluation analysis also performed by RIVSIM.

ECONOMIC ANALYSIS

Performance information alone does not offer the analyst clues as to "how expensive" it might be to improve performance at a given node or nodes. However, the RIVSIM model was set up with many sophisticated economic accounting schemes designed to help provide such useful planning information. Figure 11 illustrates the economic analysis output for an individual M & I node. Potential benefits are computed according to some preset "value" associated with satisfying the total M & I demand at the given node. Shortage losses are incurred whenever target demands

are not met-- the shape of the loss function is non-linear and node-specific. Actual benefits are computed as potential benefits minus shortage losses. Remaining benefits are computed by subtracting from the actual benefits the "attributable" portion of the project costs which regulate stream flow and thereby supply the M & I water.

The precise details of all these calculations are described elsewhere (References 5 and 11). Again, M & I nodes are among the most simply handled in RIVSIM's economic analysis. For example, for power generation facilities and irrigation sites, RIVSIM is designed to generate the "internal rate of return" as well as a benefit cost ratio. The separable-cost remaining-benefit (SCRB) approach is incorporated in the model to allocate costs and benefits between water use and water supply nodes. Also, since benefits of certain types are not immediately realizable, and the simulation period is often over several years, RIVSIM is designed to allow for a build-up of benefits (and similar changes in O&M and capital expenditure). All in all, RIVSIM evaluates all individual system components-- in economic terms-- as if they are simultaneously operated as part of a single system. This is important because, as already stated, only in this way can the true economic advantages of reuse be demonstrated.

OUTPUT AGGREGATION

The discussion above focuses on RIVSIM output regarding individual system components. However, it is worth noting that the model is designed to permit various types of output aggregation. For example, the entire system and/or several sub-systems can be analyzed in performance and economic terms. Applying this idea to a site-specific reuse simulation would permit, say, the performance and economics associated with a given category of users and/or suppliers to be examined.

MULTIPLE OBJECTIVES

One last feature of RIVSIM is also noteworthy. Recent advances in public expenditure theory suggest that criteria of economic efficiency alone do not provide a sufficient basis on which to evaluate and plan systems which are to be paid for with public funds (Reference 6). Multiple-objective water resource planning is theoretically more appropriate as discussed by Major (Reference 7). With this in mind, RIVSIM was designed to be able to accommodate multiple-objective planning. For instance, with respect to irrigation areas, in addition to economic output, the model can report on labor requirements associated with given acreage under various cropping patterns. When planning objectives place a high priority on agricultural employment, this sort of output can be a significant aspect of the simulation result. Reuse, too, should be approached from a multiple-objective framework and simulation models designed with this in mind.

CONCLUSIONS

Overall, the analysis of reuse potential as part of larger water/waste-water system can be addressed with existing types of systems analysis tools. In adapting these tools to include reuse, their inherent limitations and strengths must be kept clearly in mind. Moreover, much of what has been learned from the application of systems methods to water resource planning in general is directly applicable to planning for systems which include reuse.

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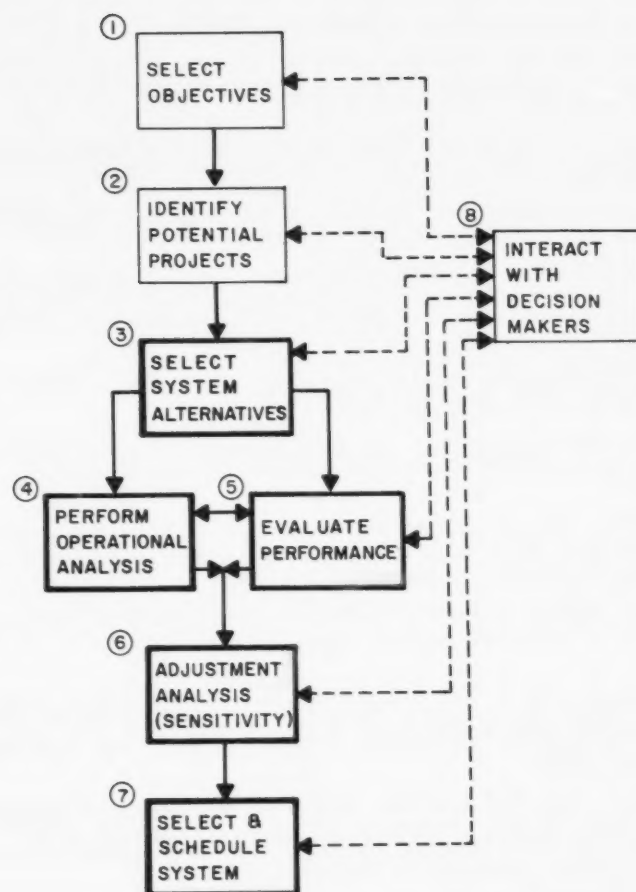


Figure 1
**STEPS IN THE EXECUTION OF A WATER
RESOURCE SYSTEM PLANNING EXERCISE**

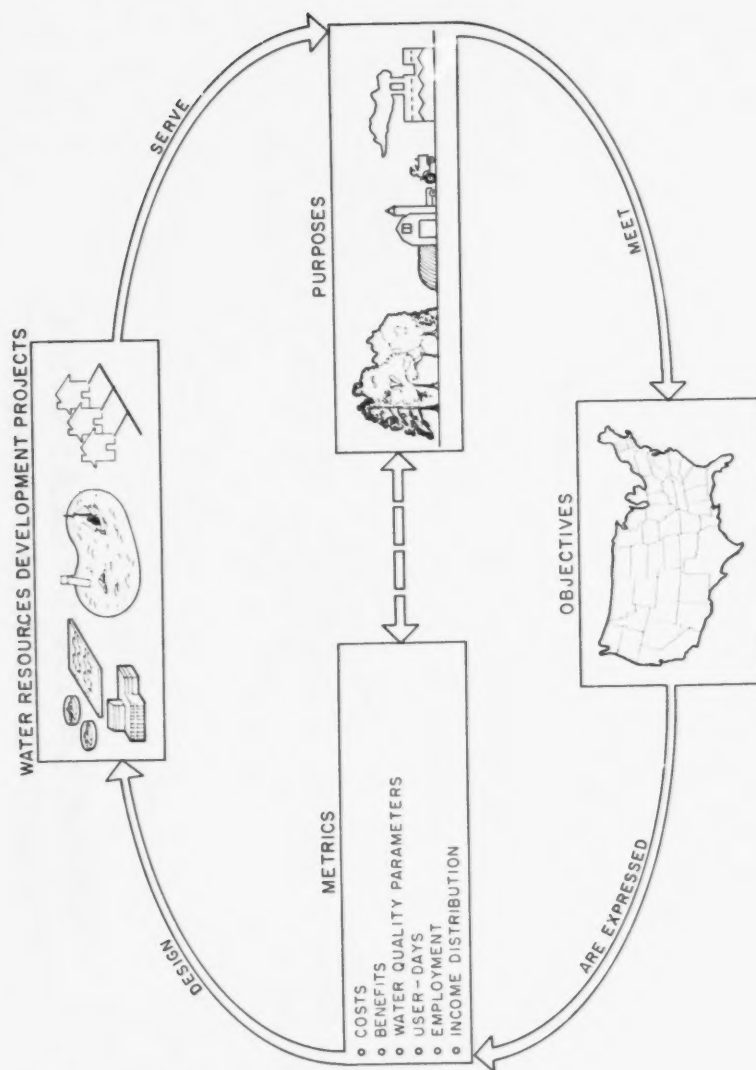


Figure 2
A NORMATIVE MODEL OF THE DESIGN OF WATER
RESOURCE SYSTEM DEVELOPMENT PROJECTS

$$\text{MINIMIZE COST} = \sum_{i=1}^m \sum_{j=1}^n c_{ij} x_{ij}$$

SUBJECT TO

$$\sum_{j=1}^n x_{ij} \leq s_i \quad \text{for } i = 1, 2, \dots, m \text{ (supply)}$$

$$\sum_{i=1}^m x_{ij} \geq d_j \quad \text{for } j = 1, 2, \dots, n \text{ (demand)}$$

$$x_{ij} \geq 0 \quad \text{for all } i \text{ and } j$$

SOURCES

SURFACE SUPPLIES
GROUND WATER
REUSED WATER ETC.

USERS

INDUSTRIAL
DOMESTIC
AGRICULTURAL ETC.

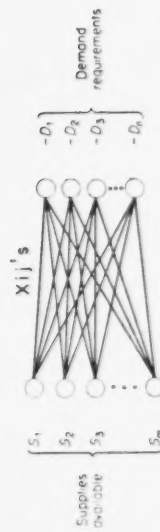


Figure 3
FORMULATION OF CLASSICAL
"TRANSPORTATION PROBLEM"

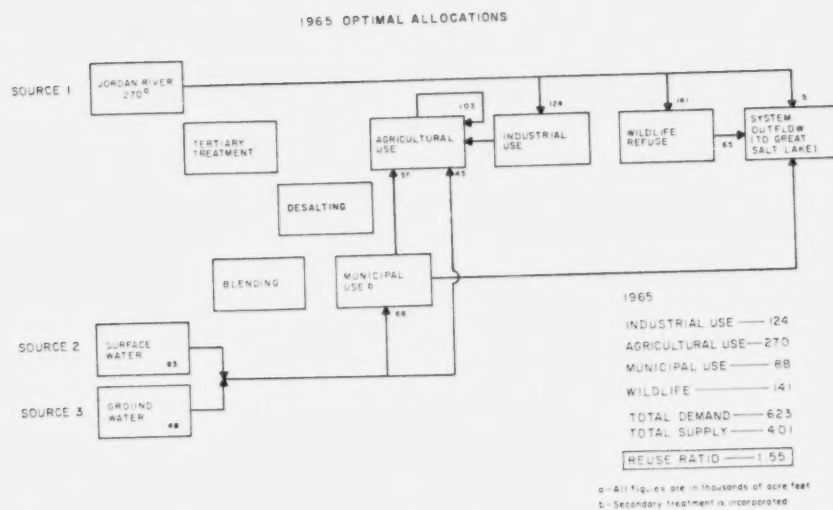
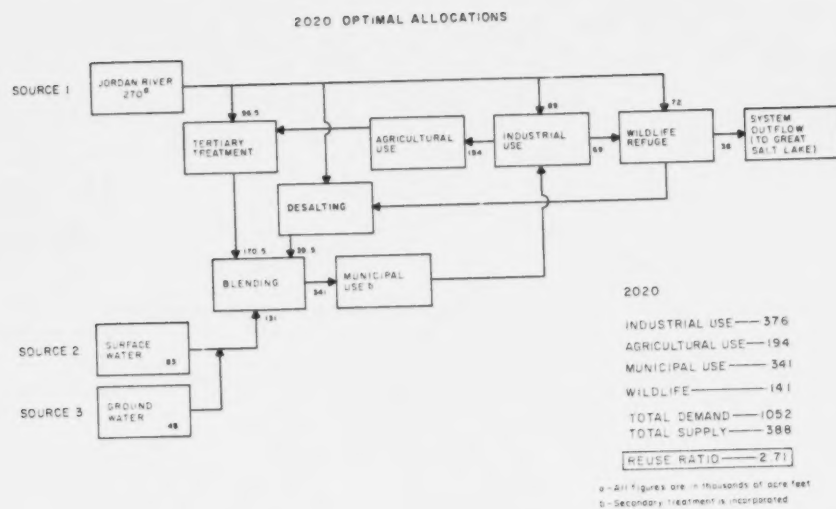


Figure 4
THE OPTIMAL WATER SUPPLY ALLOCATIONS FOR SALT LAKE CITY UNDER GIVEN AND FUTURE CONDITIONS
(after Bishop & Hendricks, 1971)



Figure 5
HYPOTHETICAL RIVER BASIN LAYOUT

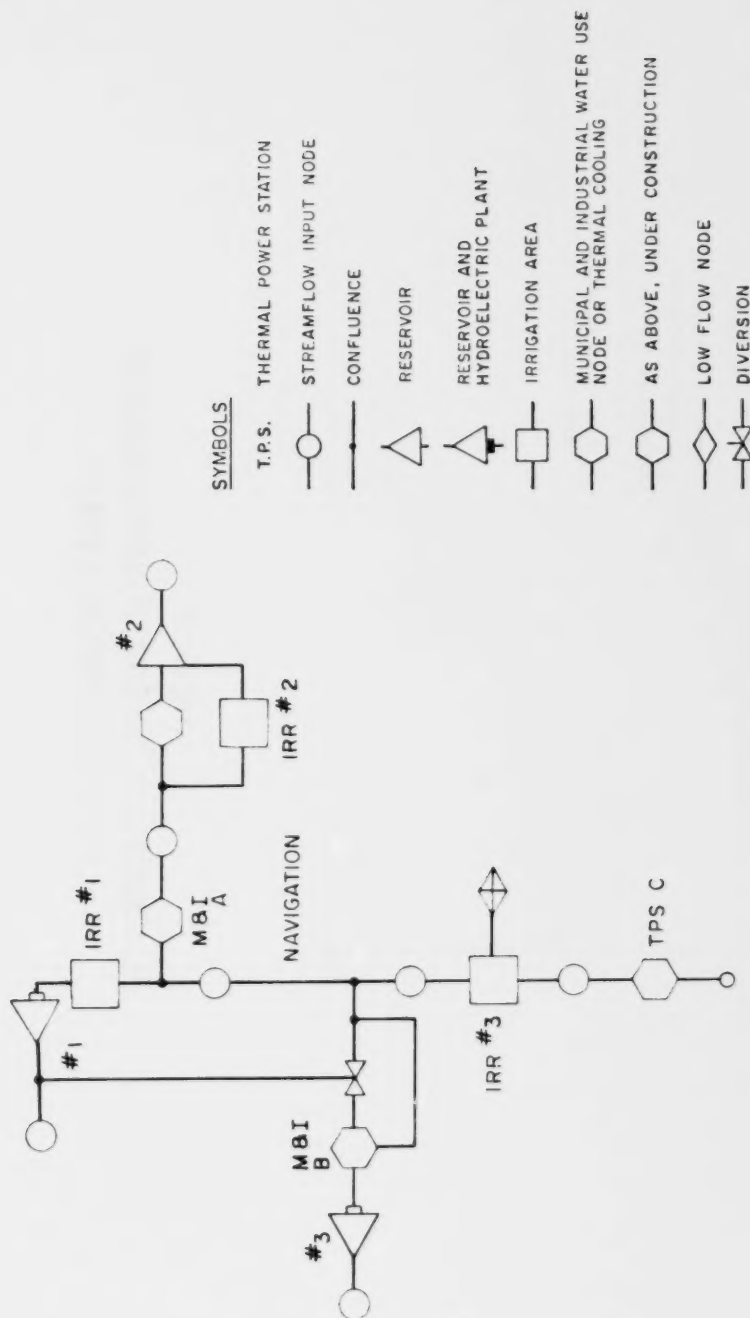
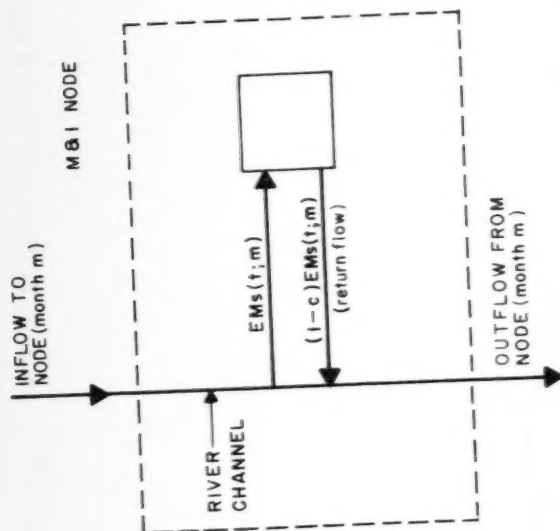


Figure 6
SCHEMATIZATION OF HYPOTHETICAL RIVER BASIN
FOR RIVSIM SIMULATION PURPOSES



WHERE

$EMs(t, m)$ is the actual diversion for M&I use during month m of year t

c = consumptive use coefficient input to the model

Figure 7
RIVSIM'S PRESENT APPROACH
TO M&I SIMULATION

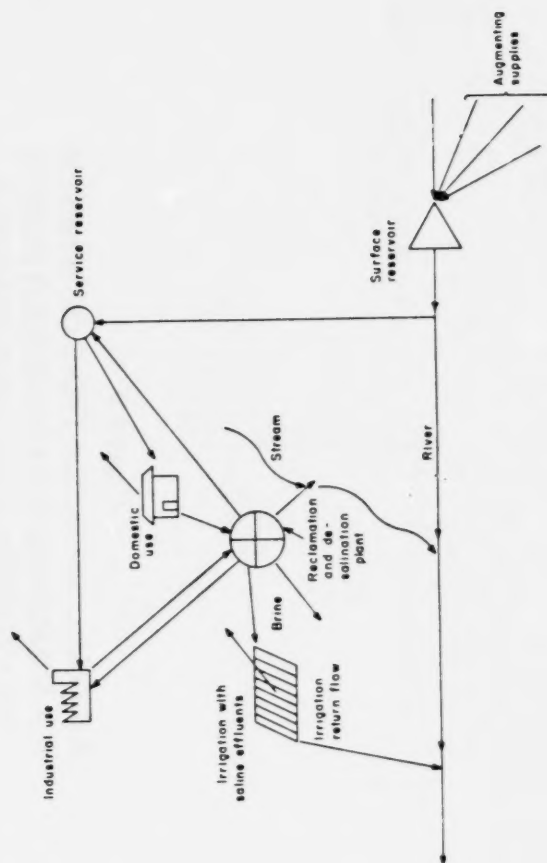


Figure 8
POSSIBLE REPRESENTATION OF REUSE
FOR SIMULATION PURPOSES

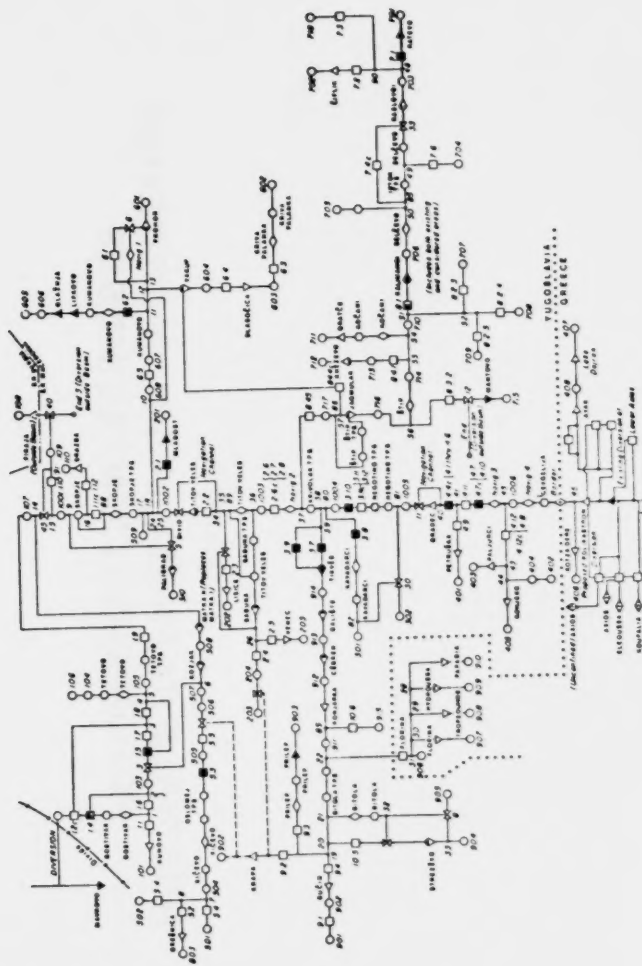


Figure 9
ACTUAL APPLICATION OF RIVSIM
TO VARDAR/AXIOS RIVER BASIN
IN YUGOSLAVIA AND GREECE

PERFORMANCE EVALUATION FOR SURFACE WATER SUPPLY FOR MUNICIPAL AND INDUSTRIAL USE AT:
PRILEPMI

GENERAL CHARACTERISTICS

MAXIMUM POTENTIAL DIVERSION CAPACITY..... = ***** M**3/SEC
DIVERSION CAPACITY..... = ***** M**3/SEC
TOTAL DEMAND..... = 4.00 MCM/YR
CONSUMPTIVE USE PERCENTAGE..... = 10.00

MONTHLY PERFORMANCE

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	YEAR
TARGET DEMAND (MCM)	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	4.0
RELIABILITY (TIMES TARGET MET)	89.	100.	100.	100.	100.	100.	89.	74.	74.	68.	74.	84.	98.
AVERAGE DEFICIT (MCM)	0.01	0.0	0.0	0.0	0.0	0.0	0.02	0.06	0.07	0.07	0.05	0.04	0.3
MEAN DIVERSION (MCM)	0.32	0.33	0.33	0.33	0.33	0.33	0.31	0.28	0.26	0.27	0.28	0.30	3.7

Figure 10
AN EXAMPLE OF RIVSIM
PERFORMANCE EVALUATION OUTPUT
FOR AN INDIVIDUAL M&I COMPONENT

BENEFIT AND COST ANALYSIS FOR MUNICIPAL AND INDUSTRIAL USE AT: BITOLA

TARGET DEMAND	=	33.87	MCM/YR
CAPITAL COSTS.....	=	0.0	\$
OMR COSTS.....	=	0.0	\$
TOTAL COSTS.....	=	0.0	\$
TOTAL ALLOCATED COSTS (see below).....	=	0.31	\$
TOTAL COSTS.....	=	0.31	\$

RESERVOIR, GROUNDWATER OR DIVERSION COSTS ATTRIBUTED TO THIS SITE:

PROJECT TYPE	NAME	TOTAL COST	% ATTRIBUTED TO M & I USE	ATTRIBUTED COSTS
		\$	%	\$
RESERVOIR	STREZEVO	35.41	1.	0.31
TOTAL	BITOLA	35.41	0.	0.31

POTENTIAL M & I BENEFITS =.....	=	7.40	\$
SHORTAGE LOSSES=.....	=	2.19	\$
ACTUAL M & I BENEFITS+.....	=	5.21	\$
TOTAL REMAINING BENEFITS++.....	=	4.90	\$
TOTAL BENEFIT-COST RATIO.....	=	16.84	\$

- = PRESENT VALUE OF ANNUAL BENEFITS IF NO SHORTFALLS FROM THE SPECIFIED SUPPLY TARGET OCCUR
- = PRESENT VALUE OF ANNUAL LOSSES DUE TO SHORTFALLS FROM SPECIFIED SUPPLY TARGET
- + POTENTIAL BENEFITS MINUS SHORTAGE LOSSES
- ++ INCLUDE ATTRIBUTABLE RESERVOIR, DIVERSION, AND/OR PUMPING COSTS WHERE APPLICABLE

Figure 11

**AN EXAMPLE OF RIVSIM ECONOMIC
ANALYSIS OUTPUT FOR AN INDIVIDUAL
M&I COMPONENT**

DESIGN/PERFORMANCE ANALYSIS OF WATER RECLAMATION FACILITIES

Raymond J. Avendt
Assistant Vice President
Consoer, Townsend & Associates
360 East Grand Avenue
Chicago, Illinois 60611

Various mathematical models are available for the design and simulation of wastewater treatment facilities. The design parameters used in these models are based on regulatory standards, equipment manufacturers recommendations, empirical relationships and the design engineer's conservatism. The simulation capabilities of the models are based on theoretical concepts, correlations between design and performance and the sophistication of the simulation subroutines. The design-performance relationship is common to both the design models and simulation models. The design is dictated by certain regulatory requirements for performance, and the facility performance is governed by the design. This assumption that the design performance equals actual performance requires validation in view of escalating capital and operational costs.

A rigorous evaluation of performance as a function of design and operation would allow the assessment of the degree of performance obtainable from a facility and the reliability. This evaluation required the development of and extensive data base of design criteria and operations records in order to develop a design/performance analysis. The adequacy of present design criteria and simulation equations could then be verified by the design/performance analysis.

A review of the literature indicates that the use of design and simulation models has been limited to a cursory application by engineering consultants, regulatory agencies and university researchers. The most frequently used model is the EXEC by Eilers and Smith that is intended for design with limited simulation capabilities. CAPDET, developed by the Corps of Engineers, is presently the most extensive design model and is currently being expanded to represent the state of the art regarding design and simulation. These two models served as the basis for the evaluation of the effects of design and operation on a facility's performance and reliability. Additional subroutines developed for individual unit processes were taken from other available models and used where the EXEC or CAPDET subroutines were not compatible with the data base. It was determined that many of the models use the same equations to design treatment processes, simulate the performance and generate costs data. This may be a deficiency in some instances where more sensitive equations are available for simulation.

Detailed evaluations were made to select the most valid set of equations for performance and reliability. The selection was based on a correlation of existing treatment unit design and operations data with observed performance.

The development of the data base necessary for the design/performance analysis, required contact with numerous federal and state regulatory agencies, consulting engineering firms and municipal treatment plants. Care was taken to assure an adequate mix of treatment facilities in order to obtain data independent of specific operation guidelines, environmental conditions, wastewater characteristics and design parameters. The operations data from a total of 85 treatment plants were available. These data were predominately from secondary activated sludge plants greater than 1 mgd and currently meeting regulatory agency effluent discharge standards. Monthly operations records for a two year period were tabulated.

The integrity of the raw data used to develop the data base was evaluated by plotting the overall plant BOD_5 and Suspended Solids removal efficiencies. If the performance data could be fitted by a lognormal distribution, it was assumed that the facility was in statistical control and the variations in performance were the result of consistent causes. Those plants whose performance fit a lognormal distribution were assumed to represent facilities performing in response to the design and operational parameters. This assumption also dictates that any deviations in environmental conditions, wastewater characteristics, operation and maintenance were balanced out over the two year data base period. A majority of the 14 treatment facilities in statistical control were visited in order to determine the validity of the laboratory analyses. The evaluation of analytical results was used as a second test of the integrity of the data base. If the results for a facility were confirmable within the relative standard deviation and relative error for 90 percent of the influent and effluent BOD_5 , suspended solids, ammonia nitrogen and orthophosphate analyses, the data were assumed valid.

The formation of the data base terminated with the development of various subfiles. The subfiles included secondary, tertiary and sludge treatment processes. Both the effect of a process on the overall performance and reliability of a facility, and the availability of an adequate simulation model were evaluated to establish a subfile. Subfiles were developed for primary sedimentation, conventional activated sludge processes, trickling filter processes, nitrification, filtration, microscreening, flotation thickening, anaerobic digestion and vacuum filtration.

An investigation of the available methods to evaluate performance data considered data plots vs. time, relative frequency plots, normal distribution plots and lognormal distribution plots. The lognormal frequency distribution plot was selected to analyze the relationships between design, operation and performance. This method allowed

evaluation of performance on a probability basis with the geometric mean value approximated at the 50% probability (or median) level. When the data is in statistical control the geometric mean is a fractional value approaching the arithmetic mean.

An investigation of the available methods to evaluate the reliability of a process considered plotting the standard deviation of performance data, 95 percent confidence intervals, R Charts, the relative standard deviation, coefficients of variation, and the Spread Factor. The Spread Factor, which is the antilog of the standard deviation of the logarithms of the original data on the lognormal frequency distribution plot, was selected to analyze the relationships between design, operation and the reliability of performance. Since the Spread Factor represents the slope of the log normal probability plot, the performance and reliability can be determined from the same figures.

Prior to using the lognormal probability plot method to assess performance and reliability, each treatment unit process was studied to determine the design and operating parameters that are assumed to affect performance. This required a review of regulatory design criteria and recommended operational guidelines for each process. Only those parameters that could be tested for correlation using the data base were of interest. It was not feasible to consider the interval theoretical aspects of the physical, chemical or biological phenomenon used in wastewater treatment because the data base contained macroscopic input-output type data. In addition it was the intent of the work to determine if the present regulatory design and operational guidelines actually affect performance or reliability. It has long been argued by researchers and designers that the mandated design criteria are too general and conservative for universal application. Therefore design parameters such as lbs BOD₅/1000 cu.ft. of aeration tank, F/M, and surface loading rate were evaluated rather than endogenous decay rates and maximum substrate utilization rates. This approach allowed the greatest amount of data to be analyzed, since the plant operations records which were used to develop the data base contained as a minimum only the operational data for those parameters required by the regulatory agencies.

Using this macroscopic method of design/performance analysis, each treatment process was evaluated to determine its transfer functions. A transfer function is the relationship between the input and output of a treatment process. Significant changes in the transfer function in response to a different design or operating parameter indicates that the parameter affects performance and/or reliability. Extensive research has been undertaken to mathematize the transfer function relationships. These equations serve as the basis of the computer design and simulation programs. Pragmatic analysis of these transfer

functions has led to the establishment of the design and operational guidelines mandated by the regulatory agencies.

A summary of the design/performance analyses for the conventional, tertiary and sludge treatment processes is presented below and on Figure 1, according to individual treatment process:

Primary sedimentation performance and reliability was observed to be independent of the surface loading rate between the range of 750 to 1200 gal/day/ft². There was no apparent relationship between suspended solids removal and depth. The underflow solids concentration was inversely related to detention times between 3 to 6 hours. Detention times less than 3 hours did not affect performance. It is proposed that the inability to correlate variations in primary sedimentation tank performance and reliability to specific design criteria indicates that deficiencies in current design practice may be compensated for by proper operation and control.

The activated sludge process BOD₅ removal was observed to perform independently of the F/M ratio between the range of 0.25 to 0.4 and the solids retention time safety factor values between 1.25 to 2.5. The reliability was positively correlated with solid retention time safety factors greater than 1.75. The final sedimentation tank suspended solids removal appeared independent of surface loading rates between 400-1000 gal/ft²/day, and solids loadings between 20-35 lbs/day/ft². A direct relationship was observed between reliability and solids loading. An overall evaluation of the activated sludge process indicates that the data base does not contain critical design or operational data to significantly affect performance.

The design/performance analysis of trickling filter data indicated that there is no apparent relationship between performance and organic loading rate (20-50 lbs/1000 cu.ft./day). A single stage carbonaceous nitrification activated sludge system was analyzed. The percent ammonia nitrogen remaining was found to have a direct relationship with the solids retention time safety factor for the range 0.5 to 3.0. The reliability of the performance was relatively constant beyond a safety factor of 1.0.

The analysis of gravity mixed media filtration of activated sludge final effluent indicated that suspended solids removal could not be correlated with filtration rates between 2.25 to 5.0 gpm/ft². It was observed however that the reliability improved with decreased hydraulic loading.

The performance of microstrainers on activated sludge final effluent indicated an inverse relationship with suspended solids removal efficiency and either hydraulic (3.0-7.0 gpm/ft²) or solids loadings (0.7-2.2 lbs/ft²/day).

The reliability of the microstrainer performance decreases with increased hydraulic and solids loadings.

Flotation thickener performance, as percent thickened sludge, could not be correlated to solids loadings between 5 to 20 lbs/ft²/day. The reliability did not respond to variations in the solids loading.

The performance of anaerobic digestion facilities represented as percent volatile solids destruction was found to be directly related to increased solids retention times between 20 to 50 days. An upper limit (90 percent probability) of volatile solids reduction was observed at 75 percent. An evaluation of the percent volatile solids destroyed as a function of organic solids loading did not provide any discernable relationship over the range of 0.03 to 0.3 lbs VSS/cu.ft./day.

The digested sludge percent solids was analyzed for hydraulic loadings in the separation tank between 10 and 50 days. A consistent solids content was observed for all detention times greater than 10 days.

Vacuum filtration performance as measured by percent dry solids in the cake increased with increasing feed solids concentrations. The reliability of the process remained relatively stable.

The results of the design/performance analysis require prudent interpretation. The analyses of conventional, tertiary and sludge treatment unit processes were conducted using design and operational data for 14 major wastewater treatment plants operating in statistical control. This assumes that the variations in performance are the result of consistent causes such as the design and method of operation. The normal random uncontrollable variations in performance are not significant to cause a facility to be out of statistical control. The results must also be considered relevant for the range of data base values. This is important when considering that the regulatory standards are intended to be conservative. Since the facilities contributing data to the data base were designed in conformance with regulatory standards, the data may not represent levels that are critical to the transfer function.

Care was taken to assume that when design or operational parameters were grouped for analysis, that the grouping did not isolate individual treatment facilities. This would tend to bias the data and limit its application. Extrapolation of the results beyond the range or groupings of data is not valid. The performance levels and reliability indicated on the lognormal distribution plots are to be considered representative of the data base and relative for other facilities.

Although the design and operational parameters that were

able to be analyzed may be considered rudimentary, the results indicate that in many instances there is no discernable relationship affecting performance or reliability. This leads to the recommendation that the design/performance analysis method be utilized with other data bases. Confirmation of the results obtained from this work would indicate that a reevaluation of the commonly accepted design and operational parameters is warranted.

FIGURE 1 - SUMMARY OF DESIGN/PERFORMANCE ANALYSES

Process	Design/Performance Transfer Function Evaluated	Design Parameter Range Analyzed	Effect on Performance	Effect on Reliability
Primary Sedimentation	Percent SS Removal vs. Overflow Rate	750-1200 Gal/SqFt/Day	Not Apparent	Not Apparent
Primary Sedimentation	Percent SS Removal vs. Depth	8-13 Ft	Not Apparent	Not Apparent
Primary Sedimentation	Underflow Solids Conc. vs. Det. Time	1-6 Hrs	3 Hrs Not Apparent 3-6 Hrs Inverse Relationship	-
Activated Sludge	Percent BOD Remaining vs. F/M	0.25-0.4	Not Apparent	Not Apparent
Activated Sludge	Percent BOD Remaining vs. θ_c/θ_c^m	1.25-2.5	Not Apparent	Direct Relationship - ship < 1.75
Activated Sludge	Effluent SS vs. Overflow Rate	400-1000 Gal/SqFt/Day	Not Apparent	Not Apparent
Activated Sludge	Effluent SS vs. Solids Loading	20-35 Lbs/Day/SqFt	Not Apparent	Direct Relationship - ship
Trickling Filters	Percent BOD Remaining vs. Organic Loading	20-50 Lbs/1000 CuFt/Day	Not Apparent	Not Apparent
Nitrification Activated Sludge	Percent NH_4^m Remaining vs. θ_c/θ_c^m	0.5-3.0	Direct Relationship - ship	<1.0 hr. Direct Relationship >1.0 hr. Not Apparent
Filtration	SS Removal vs. Filtration Rate	2.25-5.0 Gpm/SqFt	Not Apparent	Inverse Relationship
Microstrainers	Percent SS Remaining vs. Hydraulic Loading	3.0-7.0 Gpm/SqFt	Direct Relationship - ship	Inverse Relationship

<u>Process</u>	<u>Design/Performance Transfer Function Evaluated</u>	<u>Design Parameter Range Analyzed</u>	<u>Effect on Performance</u>	<u>Effect on Reliability</u>
Microstrainers	Percent SS Remaining vs. Solids Loading	0.67-2.2 Lbs/SqFt/Day	Direct Relation-ship	Inverse Relationship
Flotation Thickening	Thickened Sludge Percent Solids vs. Solids Loading	5-20 Lbs/SqFt/Day	Relatively Constant	Not Apparent
Flotation Thickening	Thickened Sludge Percent Solids vs. Hydraulic Loading	0.1-1.0 Gpm/SqFt	Not Apparent	-
Anaerobic Digestion	Percent Volatile Solids Destroyed vs. SRT	20-50 Days	Direct Relation-ship	Direct Relationship
Anaerobic Digestion	Percent Volatile Solids Destroyed vs. Organic Loading	0.03-0.3 Lbs VSS/1000 CuFt	Not Apparent	-
Anaerobic Digestion	Digested Sludge Conc. vs. Separation Tank Det. Time	10-50 Days	Relatively Constant	Relatively Constant
Vacuum Filtration	Percent Dry Solids vs. Feed Solids Con.	2.5-8.5 Percent	Direct Relation-ship	Not Apparent Relatively Constant

INSTRUMENTATION FOR AUTOMATING WATER PROCESSING SYSTEMS

Patrick Y. Yang
Program Manager
Life Systems, Inc.
Cleveland, OH 44122

Walter P. Lambert
Sanitary Engineer Staff Officer
U.S. Army Medical Research and Development Command
Fort Detrick, Frederick, MD 21701

Barry W. Peterman
Research Area Manager for Advanced Wastewater Treatment
U.S. Army Medical Bioengineering Research and Development Laboratory
Fort Detrick, Frederick, MD 21701

J. David Powell, Jr.
Director of Product Technology
Life Systems, Inc.
Cleveland, OH 44122

INTRODUCTION

A versatile computer-based instrumentation has been developed for real-time control and monitor of water processing systems. The automated water processing system of this development program was a full-scale pilot plant for a U.S. Army field hospital water reuse system. The water processing system was designed to treat nonsanitary wastewaters of certain field hospitals for nonconsumptive reuse or to purify natural fresh or brackish waters for potable use and at the same time treat the nonsanitary wastewaters for surface discharge. The pilot plant integrates the unit processes of equalization, pre-The automatic control/monitor instrumentation unit uses state-of-the-art electronics to automate the water processing system, simplify the operational requirements, protect the system and personnel and handle the automatic data acquisition which was designed as part of this development effort. The instrumentation unit is characterized by:

- Computerized, automatic process parameter control
- Automatic operating mode control
- Automatic operating mode transition control
- Automatic setpoint modification
- Fault detection and isolation analysis
- System performance trend analysis
- Interactive operator/system interface
- Automatic data transmission to Data Acquisition System

The instrumentation unit can be used, with some modifications, to control and monitor other types of water processing or wastewater treatment systems. Because the control algorithms are implemented with software, very few hardware changes are needed when applied to a different water process train. A minicomputer was used for this development program because of its better software/hardware support than with microprocessors. In applications where instrumentation size, weight and power consumption are critical, the instrumentation

can easily be upgraded with microprocessors and other microelectronics components.

The importance of instrumentation needed to automatically control and monitor Water Processing Systems (WPS) has received increasing recognition over the past few years⁽¹⁻⁴⁾. A WPS for wastewater reuse and treatment typically requires an integration of many complex unit processes such as Equalization, Prescreening, Ultrafiltration (UF), Depth Filtration, Ion Exchange (IE), Carbon Adsorption, Reverse Osmosis (RO), Ultraviolet-activated Ozone Oxidation and Hypochlorination. As complex unit processes are integrated into the WPS, automatic Control/Monitor Instrumentation (C/M I) becomes one of the most important design considerations to provide the WPS with reliable operation, maximum performance, minimum operator skill level requirement, reduction of operator errors and simple operation of a complex system.

INSTRUMENTATION SCOPE

Although some overlaps exist, the instrumentation for automating a WPS can be divided into two portions: Control Instrumentation and Monitor Instrumentation⁽⁵⁾.

Control Instrumentation

Control Instrumentation typically includes the following functions:

- a. Regulating a physical or chemical process to maintain process parameters
- b. Switching process hardware configurations and setpoints so different operating modes can be accommodated
- c. Calculating operating data to optimize performance
- d. Implementing operational changes based upon system sensor inputs, parallel process performance or operator commands
- e. Logic function for fail-safe shutdown

In a WPS application, the procedural control functions (such as sequentially switching process hardware configurations from one operating mode to another) are equally important as the parameter control functions (such as maintaining a process parameter at its setpoint). The different operating modes are illustrated in Figure 1 for the WPS pilot plant developed by Life Systems for the Field Army medical facilities. The basic operating modes consist of Shutdown, Standby, Normal, Auxiliary and Unpowered. The actual operating modes and mode transitions are complicated by the requirement of the reuse system to process different water sources and to produce different end products. The water sources include natural fresh, brackish, and medical complex wastewaters (shower, operating room, kitchen, laboratory, X-ray, laundry and composite). The water products include nonconsumptive reuse water, potable water and surface discharge.

Monitor Instrumentation

Monitor Instrumentation is defined as providing the functions of protecting personnel and equipment against damage and diagnostics of faults. The goal of the latter is to minimize unscheduled system downtime. Monitor instrumentation can include real-time parametric data display, process performance trend analysis, process and compo-

nent fault prediction, fault detection and isolation, and the logic to provide maintenance aid instructions. Eventually, fault tolerance against instrumentation failures will be included.

Traditional Instrumentation

The traditional WPS instrumentation can be characterized by gauges, indicators, dials and switches on an operator's panel which requires constant attention from a highly trained operator to interpret gauge readings and to provide manual adjustments.

Semiautomatic Instrumentation

The basic design concept of the semiautomatic instrumentation is to automate only the process parameter control loops and alarm shutdown logic. The system will be in automatic operation with protection against damage once it reaches steady-state. The manual operations needed in the semiautomatic instrumentation are during the transitions and mode selections or when system or interface failures occur and manual adjustments of setpoints are required. The semiautomatic instrumentation can be characterized by automatic process parameter control, manual startup and shutdown sequences, manual selections of valve positions according to operating modes, automatic alarm shutdown, manual setpoint adjustments when judged necessary by the operator and an Operator/System (O/S) interface panel of switches, meters, gauges and status indicators.

Automatic Instrumentation

Fully automatic instrumentation is one that can be characterized by computerized automatic parameter control, operating mode control, operating mode transition control, setpoint modification, fault detection and isolation analysis, system performance trend analysis and an interactive O/S interface.

The automatic advanced WPS instrumentation for field application will be one that does not have rows of gauges, switches and red/amber warning lights. When human inputs are no longer required, the advanced instrumentation will be reduced to a black box with the operator's knowledge built-in and will require a minimum of O/S interface.

When human inputs are still required, for example, during the development phases of a WPS, the most important aspect of the instrumentation design is the O/S interface. An automatic C/M I with an interactive O/S interface during the pilot plant phase, for example, will save developmental time and resources of the program. This interface can include visual, audio and touch sensors. Such an interface in an advanced instrumentation design utilizes an intelligent operator's console. It consists of operator and system Input/Output (I/O) devices. For example, a keyboard to input commands and a video display unit to output the system messages and verify the input commands. This type of O/S interface is convenient, effective and has lower development costs and risks.

When operator inputs are still used to run or characterize the process, for example, during off design testing, the validity of the operator's commands can constantly be checked by a properly designed instrument against preprogrammed allowable commands. All invalid commands can readily be rejected to protect the system.

Automation Options

The designer of instrumentation for automating a WPS has the options to implement the C/M I utilizing programmable controllers, microprocessors, microcomputers, minicomputers and large scale computers. These options have been evaluated previously. Large volume production of instrumentation whose design is based on microprocessors can save hardware size, weight, power, volume and cost. At this time, however, the vendor development support for microprocessors is not adequate and the need for achieving the ultimate packaging objectives of a WPS instrumentation is not a current priority. In the mean time, the minicomputers of today provide the capability of the microcomputers of tomorrow. Using minicomputers instead of microcomputers is a cost effective approach to implementing the instrumentation for automating water processing systems presently. This option allows the design engineers to enjoy the benefits of the well established minicomputer hardware and software development tools today and readily transfer the results into microcomputers of the 1980's.

Commonality

Although different WPS designs use different combinations of unit processes, their automatic instrumentation designs are very similar as illustrated in Figures 2 and 3. Figure 2 shows the automatic C/M I of a full-scale pilot plant of the U.S. Army Field Hospital Waste Water Reuse System. Figure 3 is the automatic C/M I of the Vapor Compression Distillation Subsystem (VCDS) developed by Life Systems for the water recovery from urine in an aerospace application. Many of the design concepts and hardware structures are common. The major differences are in the software programs and in the power/signal conditioning modules. Each system has its own dedicated O/S interface panel with a common layout design as shown in Figure 4.

Common Hardware Elements

The common elements of the automatic C/M I hardware are shown in Figure 5. They consist of:

- a. Signal conditioning circuits for process sensor inputs
- b. Power conditioning circuits for process actuator outputs
- c. Analog and digital interfaces between the computer and the conditioning circuits including multiplexers, digital to analog converters and analog to digital converters
- d. Computer Central Processing Unit (CPU)
- e. Memory to store control programs
- f. O/S Interface
- g. C/M I to Data Acquisition and Reduction System (DARS) and Test Support Accessories (TSA) Interface when required

The last two items (f and g) are generally required during developmental phases. They become less important as the C/M I and the mechanical system reach their maturity.

Common Software Elements

The common software elements of a WPS C/M I are shown in Figure 6. They consist of:

- a. Executive program which controls the task schedule of all software programs except those which are interrupt-driven

- b. Operating mode and mode transition control programs
- c. Process parameter control programs
- d. Fault detection and trend analysis programs
- e. I/O programs
- f. Power-failure control programs
- g. Front panel command handling program
- h. DARS command/data handling programs

Again, the last two items (g and h) are more important during the developmental phases than in the field application phases.

INSTRUMENTATION DESIGN

This section presents the automatic C/M I design for a transportable U.S. Army Field Hospital WPS pilot plant⁽⁶⁾.

The automatic C/M I was based on a 16-bit minicomputer with 16K words of core memory. It was designed to interface with 64 analog inputs, five analog outputs, 128 digital inputs and 80 digital outputs. The packaged unit has dimensions of 53 cm deep, 53 cm wide and 72 cm high (21 in., 21 in. and 28.5 in., respectively). It weighs about 110 kg (250 lb) and consumes approximately 750 W of electrical power.

The mechanical hardware which the C/M I controls has six unit processes as shown in Figure 7. A descriptive flow diagram is shown in Figure 8.

The WPS has two primary modes of operation: the Reuse mode and the Potable/Discharge mode. In the Reuse mode the WPS treats and purifies nonsanitary hospital wastewaters for nonconsumptive use. In the Potable/Discharge mode it simultaneously treats wastewaters for discharge into the environment while treating natural fresh or brackish water for potable use. In addition to these two primary modes of operation, there are two secondary modes and four auxiliary modes. The secondary modes are Potable only and Discharge only. The auxiliary modes are those designed for scheduled maintainance -- UF Cleaning, IE Regeneration, RO Cleaning/Coating and System Drain. The pilot plant was instrumented with 86 electrical actuators and 106 electrical sensors in addition to mechanical gauges, pressure regulators, hand dials and other manual adjustments. A more detailed WPS pilot plant description and discussion is given in another paper entitled "Multipurpose Water Treatment System Development"⁽⁷⁾.

Design Considerations

The WPS pilot plant automatic instrumentation design considerations include the evaluation and the selection of the following areas:

- Available approaches
- Architectural options
- I/O Design Options
- Available command/data entry techniques
- Available system display techniques
- System protection considerations
- Operation simplicity considerations
- Instrumentation packaging considerations

These considerations are further expanded in Table 1.

Another important aspect of an automatic C/M I for water processing is the sensor monitoring technique selection and sensor field hardware selection criteria. These criteria are shown in Tables 2 and 3.

Operator/System Interface Design

One of the major benefits of the automatic instrumentation for the WPS pilot plant is the O/S interface panel which simplifies the complex WPS pilot plant operation. Figure 9 is a photograph of the O/S interface panel. The O/S interface panel of the automatic instrumentation unit can be divided into two portions: the control panel and the monitor panel.

Control Panel

The control panel, on the righthand side of the picture is used to select water sources, water products, auxiliary modes and system operating modes. The system command pushbuttons are designed for the operator to control the WPS system modes. As shown in Figure 1, the WPS has four system operating modes (Unpowered, Shutdown, Standby and Normal) and seven programmed allowable mode transitions. Direct transition from the Shutdown mode to the Normal mode is not allowed. In addition to the programmed allowable transitions, it is possible for the WPS to go through the Unpowered mode at any instant in time because of a power failure.

There are five product/source operating modes defined as follows:

- Producing potable water from natural fresh or brackish water sources
- Treating shower, operating room or laundry wastewaters for surface discharge
- Treating hospital composite laboratory, X-ray, kitchen or unknown wastewaters for surface discharge
- Processing kitchen or shower wastewaters for nonconsumptive reuse
- Processing composite, laboratory, X-ray, operating room, or unknown wastewaters for nonconsumptive reuse.

There are four auxiliary operating modes for the WPS maintenance. They are designed for UF module cleaning/coating, RO module cleaning/coating, IE module regeneration and system water drain.

There are three light emitting diodes (LED) on the front panel for the indication of mode transitions, override on and command disabled. A lamp-test pushbutton is also provided for maintenance.

Mode transitions are indicated by both the LED indicator and the color coded system command pushbuttons. The override-on LED is lit when any of the manual override switches located in the recessed manual override panel are used to override automatic instrumentation actuator control.

The command disabled LED is lit when the front panel pushbuttons and the operator command keyboard are disabled which is accomplished by activating a switch at the rear panel. This is designed to prevent any unauthorized or accidental activation of any pushbuttons or keyboard switches.

Monitor Panel

The monitor panel, on the left-hand side of Figure 9, is used to communicate with the system for parametric data examination, system message display, system status summary and for control/monitor and setpoint modifications.

The System Status Summary indicators are designed to give a quick summary of the system status at a glance. Detailed fault detection and isolation analysis (FDIA) messages are displayed on the CRT display unit.

The operator command keyboard consists of 40 switches. They are divided into four different groups as follows:

- Five operations -- Examine, Modify, On-line Display, Clear and Next Display
- Five functions -- Present Reading, Scale Factors, Setpoint, Allowable Range and Sequence Timing
- Fifteen sensors/actuator types -- Conductivity, Chlorine, Dew Point, Flow Rate, Hardness, pH, Pressure, Temperature, TOC/COD, Turbidity, Water Level, Other Sensor, Timer, Actuator and Others.
- Fifteen data/code entries -- numbers 0 through 9, minus sign, decimal point, Reset (to clear entry), asterisk (for no change during the modification of operation), and Enter (to indicate the end of a command sequence)

Typically, a command consists of an operation, a function, a sensor/actuator type and a data/code entry. Both the command syntax and semantic validity are checked automatically when the operator enters a sequence of operator commands. For example "examine, present reading, conductivity, 400, enter" is a correct sequence of operator commands both syntactically and semantically. A sequence of "modify, temperature, 101, enter" is syntactically incorrect because the "function" command is missing. A sequence of "on-line display, present reading, turbidity, 605, enter" is syntactically correct but semantically incorrect. It is syntactically correct because the sequence consists of all the necessary elements for a complete sentence: operation, function, sensor/actuator type and data/code. It is semantically incorrect because the sensor requested "turbidity, 605" does not exist in the WPS pilot plant.

Human Engineering Considerations

The O/S interface layout reflects the engineering effort to simplify the complex operation by grouping command pushbuttons and switches according to the functions.

The color coded system transition mode display was designed so that the operator could readily tell the system transition status. The additional mode transitions LED indicator is designed so that even a color-blind person could operate the WPS automatic instrumentation.

The automatic operator command validity checking is an important feature. The types of validity checked include the following:

- Whether a transition requested is allowable
- Whether a combination of different product and source water selection is valid

- Whether an operator command is syntactically correct
- Whether an operator command is semantically correct
- Whether an operator is authorized to operate the automatic instrumentation unit
- Whether an operator is authorized to modify setpoints
- Whether an operator is authorized to modify allowable ranges
- Whether the hierarchy of setpoints prevail (meaning Caution, Warning and Alarm setpoints must be in the right order)

To protect the system from damages caused by unauthorized personnel, there are three levels of protection. The first level is a Command Disable switch located on the rear panel of the automatic instrumentation unit. This switch may be used to disable all the front panel pushbuttons and the keyboard switches for protection against unauthorized usage. The second level is the authorization code (password) requirement when using the "Modify" or the system control commands. An authorization code is a four-digit number designed for an operator to identify himself and his authorization level. When this number, entered from the operator keyboard, matches one of the preprogrammed codes in the computer memory, the operator is then allowed to perform operation for which he is authorized. The lowest level of authorization (meaning no authorization code) would allow no modifications of any modifiable parameters in the C/M I software. One level up would allow the operator to modify the setpoints. The highest level would allow an operator to modify scale factors, allowable ranges, and sequence timing constants. During the modification of setpoints, the authorized operator must enter setpoints in the right order. It is also required that the new setpoints entered by the operator must be within the allowable ranges. The relationship of the setpoints and the allowable ranges are shown in Figure 10.

CONCLUSIONS

The versatile computer-based instrumentation for automating WPS is a viable solution to controlling the complex mechanical/electrical system. The advanced O/S interface panel simplifies the complex operation of the WPS. The WPS can be operated by one single operator with minimum amount of training because of the built-in intelligence of the automatic C/M I. The use of a minicomputer-based advanced instrumentation provides flexibility in the pilot plant phase and ease of transferring the algorithms into a smaller microcomputer-based instrumentation in subsequent stages of development.

ACKNOWLEDGEMENT

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Table 1
WPS Pilot Plant Automatic Instrumentation Design Considerations

- Available Approaches
 - Hardwired
 - Programmable Logic
 - Microprocessor
 - Microcomputer
 - Minicomputer
 - Large Scale Computer
- Architectural Options
 - Distributed design
 - Hierarchical design
 - Centralized design
 - Centralized design with redundancy
- Input/Output Design Options
 - Signal conditioners packaged with mechanical units
 - Signal conditioners with automatic instrumentation unit
 - High level multiplexing
 - Low level multiplexing
 - No multiplexing
- Available Command/Data Entry Techniques
 - Custom-made keyboard
 - Off-the-shelf 51-key typewriter-type keyboard
 - Off-the-shelf point-of-sale type keyboard
 - Thumbwheel switch/pushbutton switch combination
- Available System Display Techniques
 - Black and white CRT display
 - Color CRT display
 - Gas discharge dot matrix display
 - Four level indicator light display
 - Illuminated graphic or message display switches
- System Protection Considerations
 - Front panel command disable provision
 - Operator identification password requirement
 - Command validity check
 - Built-in allowable ranges for setpoint modifications
 - Trend analysis with system shutdown upon alarm conditions
- Operation Simplicity Considerations
 - Divisions of system command, product/source selection and auxiliary modes
 - Definition of allowable mode changes
 - Definition of allowable combinations of modes
 - Indication of mode transition
 - Description of water sources
 - Division of manual override switches by unit processes and by type of actuators
- Instrumentation Packaging Considerations
 - Modular design of electronic circuits and assemblies for ease of maintenance
 - Mounting of the CRT display unit inside the cabinet for both equipment and personnel safety
 - Removable side panels, rear panels, top panel and front panel for ease of maintenance
 - Interchangeable computer front panel: (1) plain cover, or (2) programmer's panel
 - Perforated bottom panel and louvered rear door for ventilation

Table 2
Sensor Monitoring Technique Selection Criteria

Absolute Requirements

Adequate Monitoring Range
Adequate Sensitivity and Stability
Avoidance of Interferences
Adequate Speed of Response

Primary Considerations

Continuous versus Cyclic Analysis
Direct versus Indirect Indication
Sample Size
Reagent Requirement (when required,
high reagent stability)
Degree of Specificity
Automation Possibility
Skill Level
Environmental Conditions
 Temperature
 Flow Rate
 Pressure
 Reagent Storage (if required)

Secondary Considerations

Reagent Volume
Analysis Time Sensitivity
Output Units
Sample Concentration Needed

Table 3
Field Hardware Selection Criteria

Absolute Requirements

High Safety Level
Applicable to Field Use
Adequate Operating Life
Adequate Functional Capability
Low Cost
R&D (Minimize Development Risk)
Capital (Original Equipment)
Operating

Primary Considerations

Weight (Hardware, Spares and Expendables)
Power Requirements
Volume (Hardware, Spares and Expendables)
Low Logistical Support Requirements
Expendables
Spares
Durability
Reliability
Transportability
Operation
Storage
Maintainability
Operator Skill Level and Time
Minimum Servicing
Easy Replacement
Easy Repair
Maximum Time Between Servicing
Low Noise Level
Simplicity (Operation and Configuration)
Quantity of Heating
Quantity of Cooling (Heat Rejection)
Availability of Equipment in Marketplace
Number of Different Components
Component Development Risk

Secondary Considerations

Growth Potential
Minimum Service Interfaces
Flexibility to Other Available Reagents
Weight of Individual Components

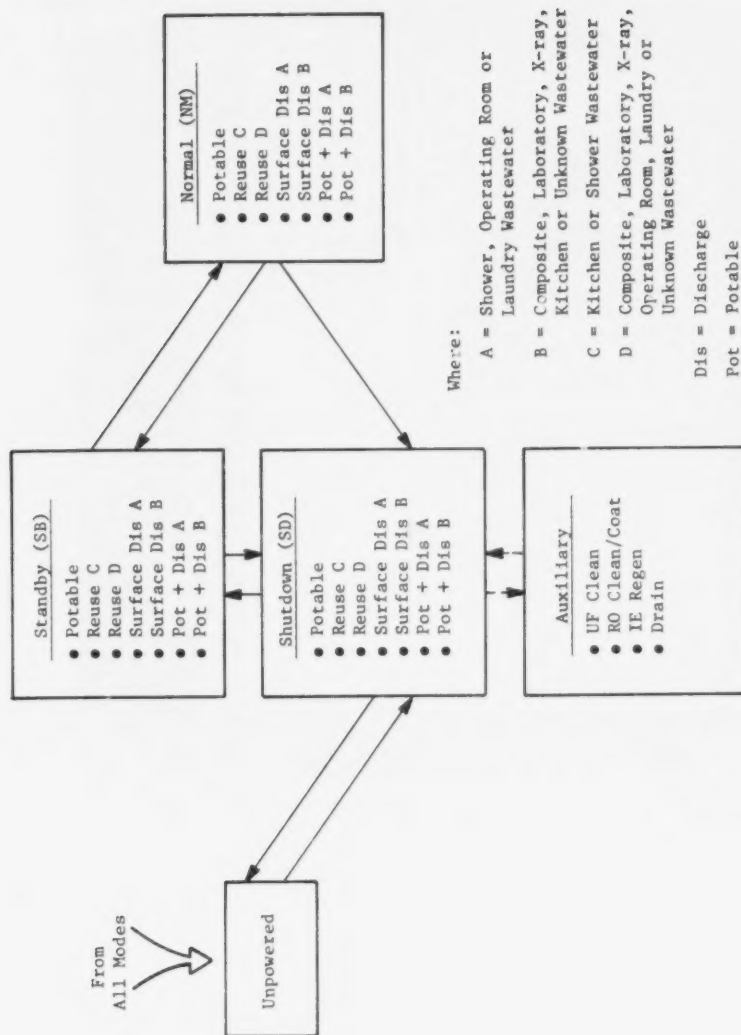


Figure 1 WPS Operating Mode/Mode Transition Block Diagram

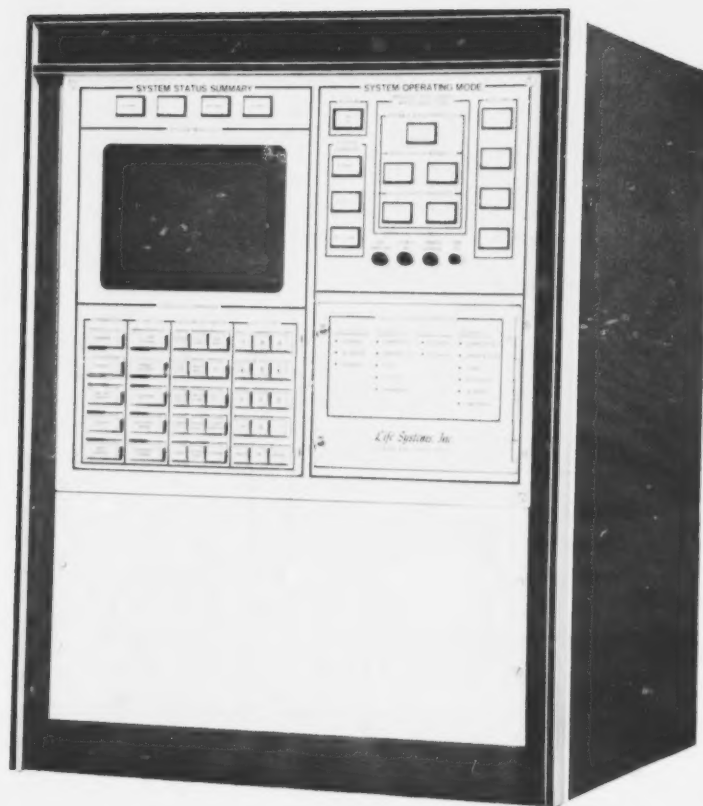


Figure 2 Automatic WPS Control/Monitor Instrumentation



Figure 3 Automatic VCDS Control/Monitor Instrumentation

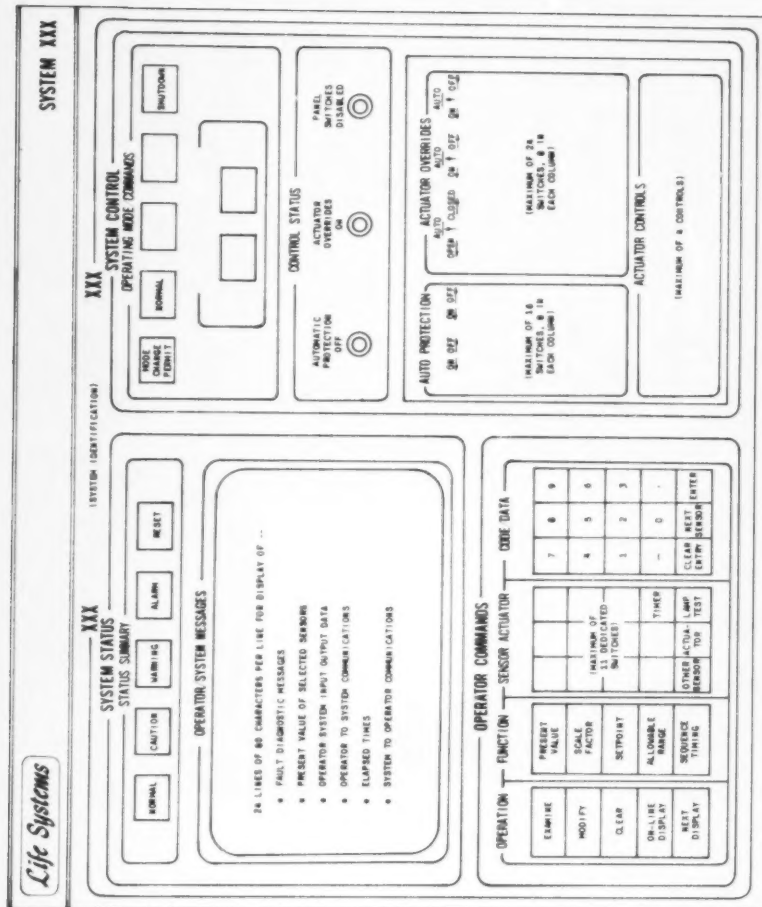


Figure 4 Common Operator/System Interface Panel Design

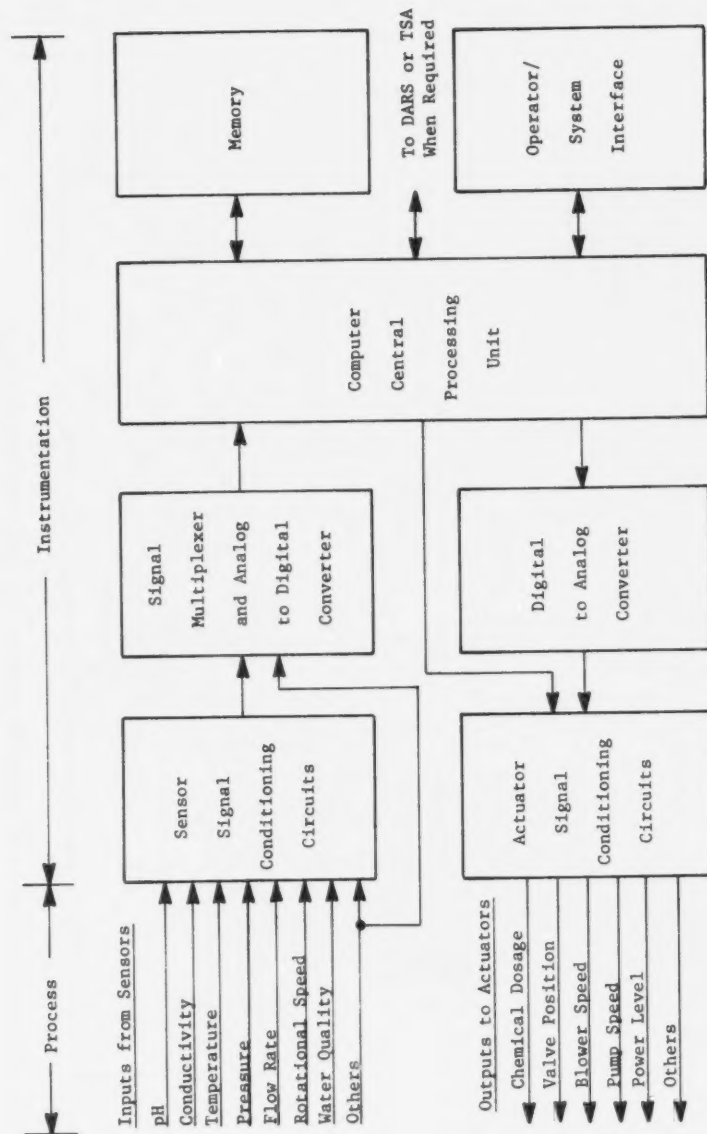


Figure 5 Common Elements of Automatic WFS C/M I Hardware

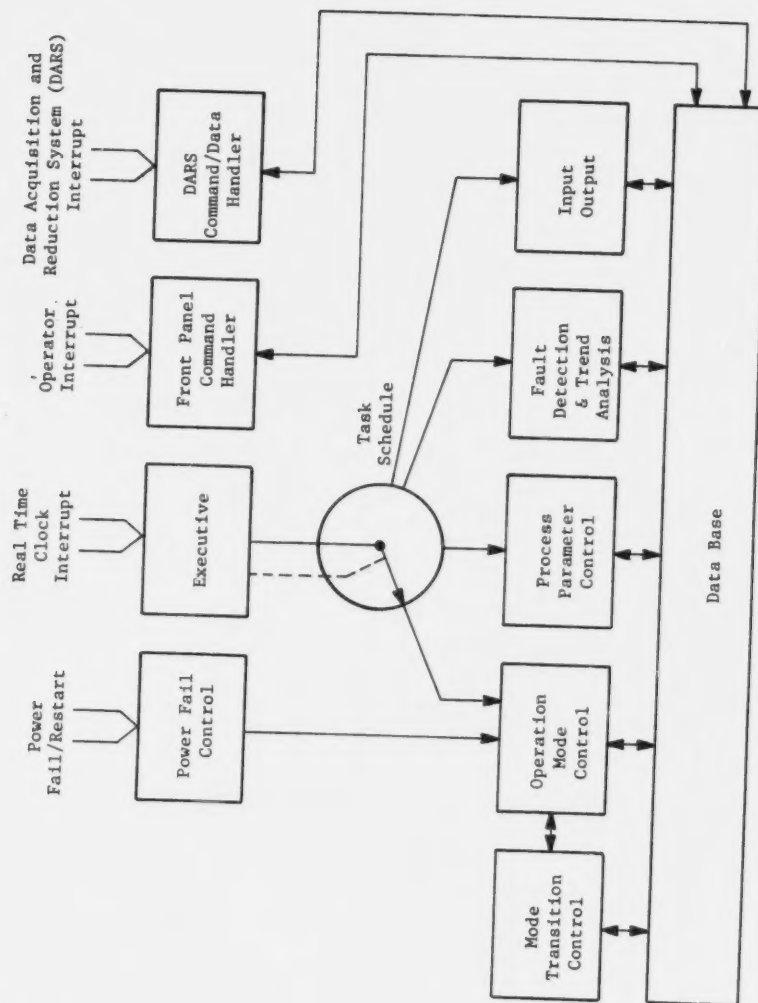


Figure 6 Common Elements of Automatic WPS C/M I Software

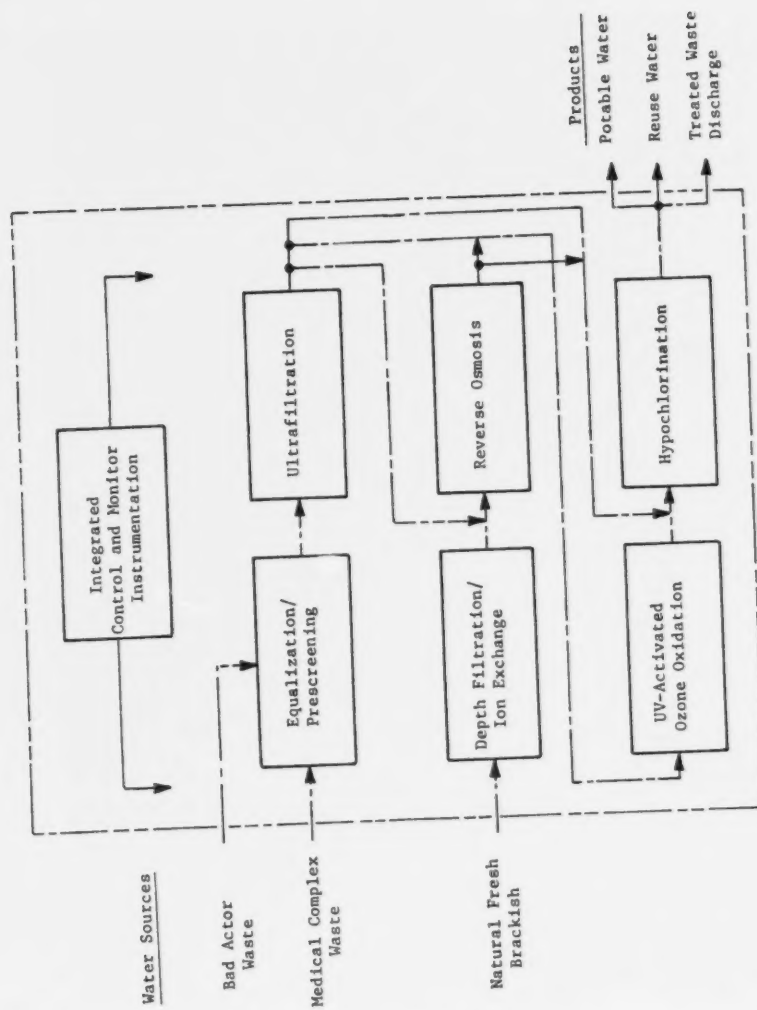


Figure 7 Water Processing System Block Diagram

[illegible]

CLEVELAND, OHIO

Figure 8 Water Processing System Flow Diagram

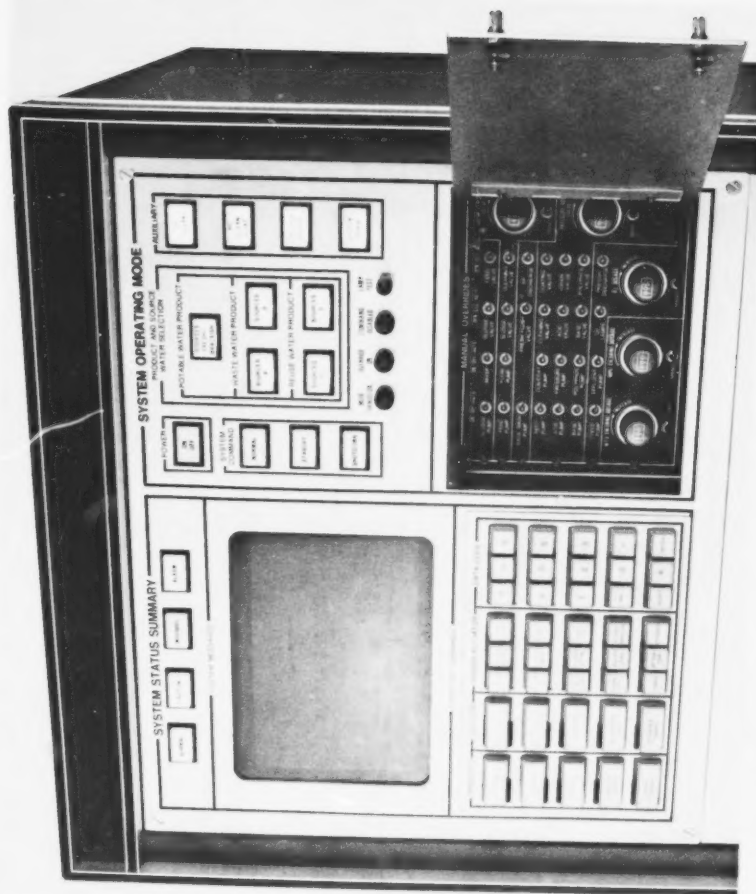
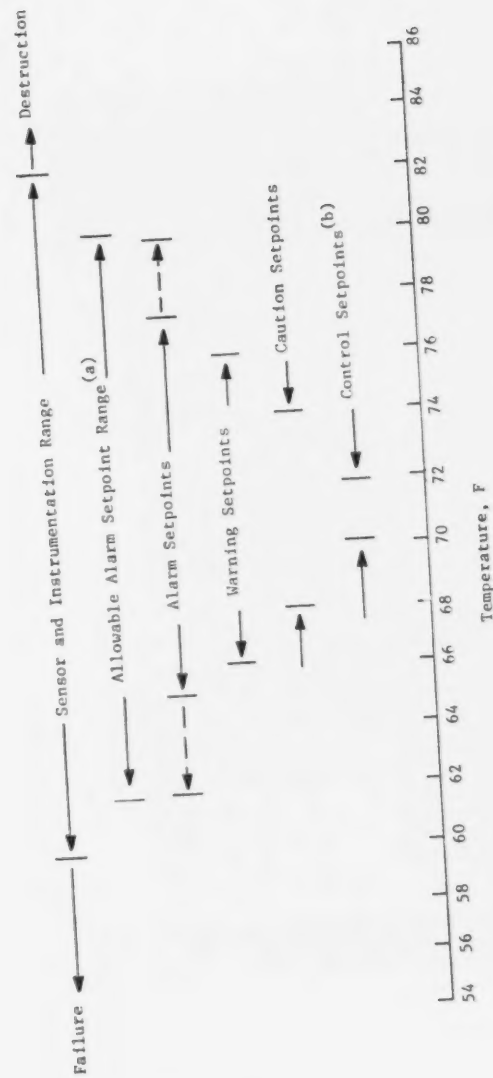


Figure 9 Automatic Control/Monitor Instrumentation Front Panel and Recessed Manual Override Panel



(a) Software imposed limit on allowable modifications.
 (b) Passwords needed to change setpoints -- different level, different authorization code.

Figure 10 Monitor Setpoints and Their Relationship

WASTEWATER REUSE AND WATER CONSERVATION AT U.S. ARMY INSTALLATIONS

Stephen P. Shelton, Ph.D., P.E.
Professor of Engineering
University of South Carolina
Columbia, South Carolina 29208

Curtis J. Schmidt, P.E.
S.C.S. Engineers
4014 Long Beach Blvd.
Long Beach, California 90807

Ernest V. Clements, III, P.E.
S.C.S. Engineers
4014 Long Beach Blvd.
Long Beach, California 90807

INTRODUCTION

Wastewater reuse has become a viable alternative water resource in the abatement of a number of stressing environmental problems. It offers a new means of extending water supplies and reducing fresh water demands. In regions where high sewer surcharge fees or stringent discharge requirements exist, reuse has the potential to reduce treatment costs by reducing or eliminating the discharge. As our water resources become stressed and pollution potential increases, effective low energy resource conservation methods must be explored; wastewater reuse is one such method.

The Army has become interested in water reuse systems because many of its industrialized bases, in the southwest and western United States, are located in water short regions. Since most existing full-scale water reuse programs in the United States are oriented towards irrigation, these facilities provided little background data for industrial reuse systems. For this reason, the Army Medical Bioengineering Research and Development Laboratory initiated a research program to develop water reuse and conservation systems for application at Army installations. It was the purpose of this investigation to adapt, test, and apply a generalized computer oriented water reuse and conservation model that would aid in the selection of the most cost-effective water use alternative at most Army installations.

BACKGROUND

The U.S. Army's primary mission is to secure our national defense. In the accomplishment of this mission, they are also charged with maintaining the quality of the national environment and complying with federal and state regulations as related to the environment. As a part of their environmental sciences research program, the Army is encouraging water conservation and reuse. One means of achieving this is through wastewater treatment and reuse.

Wastewater reuse can have several benefits for Army posts: fresh water supplies can be conserved by substituting the reclaimed water for subpotable uses; problems with pollution control can be alleviated by internal recycling and reuse at specific activities; treatment performance can be enhanced by reusing water and reducing the hydraulic load on the treatment plant; nutrients in the wastewater can be utilized as fertilizer in irrigation waters; zero discharge reuse schemes can eliminate

the problem of routine stringent effluent or pre-treatment criteria; and by reducing wastewater through the reuse, bases can reduce the cost of sewer discharge fees.

Another benefit of reuse is that it highlights the need for source reduction of waste and for proper handling and disposal of industrial residues and wastewaters. Elimination of strong wastes from the general waste stream enhances reuse potential and eases pollution abatement problems. To date, reuse at fixed Army posts has been considered only when a specific problem has forced authorities to evaluate reuse as one potential solution. The recent passage of amendments to PL-92-500 (PL-95-217) will increase this type of activity since consideration of wastewater reuse, as one alternative, is mandated by the legislation.

However, the Army still has no overall method of assessing the potential for reuse throughout its installations. The reevaluation model discussed herein provides a systems approach to assessing potential water reuse at all Army fixed facilities. The model is directly applicable to those installations where new water and wastewater construction is planned and, thus, formal evaluation of reuse potential is federally mandated. The considerations contained herein are limited to subpotable reuse and are concerned only with permanent, fixed Army posts; furthermore, installations that are concerned with storage or specialty operations, such as Aberdeen Proving Grounds and Walter Reed Hospital, or those classified as ammunition depots are not considered by this investigation.

OBJECTIVES

To develop an effective water reuse and conservation model, it is necessary to know the acceptable water quality of various industrial processes, the degradation of water quality across the various processes, and water usage for each process as a function of time. With these data reuse and conservation models can be applied to provide adequate blending, storage and treatment to fulfill the industrial system usage and quality requirements. Thus, the objective of this project was to develop a tool that could be used by the Army in the assessment of potential water reuse activities at their fixed installations and also to identify those bases with the best potential and to evaluate a conceptual reuse system at those installations. In meeting these objectives, Army activities were to be identified as to their potential for water reclamation as a function of their characteristic flow and quality. In addition, a reuse evaluation model was developed that incorporated a three-tiered approach which will be discussed subsequently.

The adaptation of a generalized water reuse and conservation model was accomplished in a four-phase approach: 1) data acquisition; 2) model adaptation; 3) sensitivity analysis; and 4) application. The data acquisition phase included a comprehensive literature evaluation of existing water reuse conservation systems, requisite water quality for industrial processes, and through-process water degradation. In addition, specific field data was obtained at three Army installations that were to be used subsequently for sensitivity analysis.

A model development phase included the development of the required flow, volume, time, and quality algorithms required to interrelate the industrial processes of concern. Once the engineering relationships were developed, the cost functions were developed for system benefit/cost analysis. These algorithms were incorporated with a methodology into a generalized water reuse and conservative model. This model was designed as a three-tiered evaluation to aid the Army in evaluating reuse poten-

tial at fixed installations. Since there are over 130 installations of concern in the United States, the first tier, by necessity, had to be a simple, straightforward precursor type model that could be used to assess the installation's overall reuse potential with about one man-day of effort. Tier II will be applied in installations that score well on Tier I. Tier II provides a cookbook-type approach, leading the evaluator to an eventual comparison of total cost for various reuse scenarios, including non-reuse options, at each installation. Tier III is a detailed decision-making model that incorporates a sophisticated mathematical model supported with a computer program to aid Army engineers in the selection and cost of conceptual reuse systems at installations with the best reuse potentials. The third phase of the investigation included a model application at three Army installations to determine the sensitivity of the parameters considered by the model. This phase permitted model tuning so that input factors indicated their true impact on the system. This is perhaps one of the most important stages in the development of any projective model.

The final phase, application of the model to an Army installation and subsequent construction of the reuse facility, is currently under consideration.

DISCUSSION OF TIER I--PRECURSOR MODEL

The purpose of the precursor model is to provide an efficient model to quantify the potential for wastewater reuse at Army facilities. Tests have been designed so that an individual, familiar with the technique, can complete an analysis of an Army installation in approximately one day, oftentimes without having personally visited the site. The result of this evaluation, using the Tier I criteria, should eliminate a substantial number of Army installations that have little potential for reuse, thereby effectively reducing the cost of analysis for Tier II and Tier III, the more complete reuse models.

The Tier I Precursor Model was developed using a paired-comparison waiting approach. Initially, five general categories or criteria were developed as being indicative as reuse potential at fixed Army installations. Subsequently, the major criteria were rank ordered as to importance for wastewater reuse as follows: water supply, wastewater, activities, institutional aspects, and climate, from high priority to low priority. In order to weigh the relative importance of each criteria, a paired-comparison procedure was used. By assigning a value of 100 to the first rank criteria, each following criteria was ranked, using the Delphi technique, as a percentage of importance as compared to the criteria that immediately preceded it. Upon completion of this procedure, it was determined that the relative importance of each category expressed as a weight-percentage of the whole were as follows: water supply, 25%; wastewater, 25%; activities, 20%; institutional aspects, 18%; and climate, 12%. In order to reflect these weightings, the best score--in terms of amenability to water reuse--would be reflected in each of the five categories of the Tier I precursor model by attainment of the maximum total possible score. It was deemed unnecessary to carry the paired-comparison approach to the next level of criteria under each of the five major categories; however, a Delphi approach was used to arrive at the point values for each of these questions.

As indicated previously, the Tier I Precursor Model is a simplistic straightforward evaluation. It was developed using a closed-ended questionnaire technique and should be easily completed with relative accuracy with the very limited data base that is usually available.

Reasoning behind the selection of individual questions and the importance for each of the five categories is keyed to the understanding of the precursor approach.

WATER SUPPLY: The costs and availability of a water supply, both now and in the future, are inherent to the need for wastewater reuse. Water shortages, potential water supply problems, and large water usage are all factors that make reuse more attractive. Questions concerned with water supply in the Precursor Model are as follows:

Question 1. Is the base water supply available from a reliable source for the next 20 years?

Remarks: A negative response signals possible long-range supply problems, a plus for reuse.

Question 2. Is there possible significant depletion of the water supply within the next 20 years?

Remarks: A negative response means future planning and possible design of new water supply facilities--a good time to evaluate reuse.

Question 3. Is there a near future anticipated problem with the water supply?

Remarks: Negative response: High rating for reuse as evaluation and planning for new and additional water supplies should include reuse possibilities.

Question 4. What is the present cost of water procurement and treatment per 1,000 gallons?

Remarks: High water costs are a driving force for reuse as the economics of reuse become more attractive.

Question 5. Is there a foreseeable event that could markedly increase water costs in the near future?

Remarks: Although costs may be reasonable now, many areas are realizing increased costs as water sources are depleted and quality degrades. Future cost increases benefit reuse economics.

Question 6. Is expansion or upgrading of the water supply/treatment systems planned for the near future?

Remarks: Reuse can provide savings in reduced plant capacity. Planning should include reuse feasibility.

Question 7. What volume of water is used on the average in MGD?

Remarks: Bases using large volumes of water realize economies of scale when considering the cost of reuse facilities. High usage may also reflect extensive irrigation and/or large industrial users; both are positive for reuse.

Question 8. What is the TDS of the base water supply?

Remarks: High TDS water is less attractive for reuse because of the low salt tolerance of some vegetation and the reduced recycle potential in cooling systems. Domestic use increases TDS by 300 mg/l, thereby exacerbating reuse problems where the water supply is already high in TDS.

WASTEWATER: Management of wastewater on an Army installation is a critical component of any wastewater reuse system. If the installation has upgraded their effluent wastewater quality to meet stringent discharge requirements, the quality of the water discharged may be of excellent quality for reuse. Conversely, if the installation is currently in a situation where outdated and/or overloaded treatment facilities are unable to meet requisite quality requirements and, therefore, an update of current facilities is under consideration by virtue of the mandate in Public Law 95-217, they must consider water reclamation or wastewater reuse as an alternative within their decision-making process. Thus, when considering the overall concept of wastewater management, the treatment facility's effluent quality, effluent criteria costs, and discharge volumes are all important factors.

Question 9. Does the base treat wastewater for direct discharge to surface water or land?

Remarks: Direct discharging bases are given positive reuse ratings because of the problems associated with meeting increasingly stringent discharge requirements. Reuse is one answer for reducing plant loadings or eliminating discharges altogether.

Question 10. Does the treatment plant (sewage) presently meet discharge requirements?

Remarks: A negative response indicates a wastewater management problem for which reuse may be part of the solution.

Question 11. Will additional treatment facilities be required within the next five years?

Remarks: A positive response indicates planning, design, and construction of new facilities. Reuse could have positive cost impact, or conceivably alleviate the problem so new facilities would not be necessary.

Question 12. What quality is the plant effluent in terms of BOD (mg/l)?

Remarks: Good quality effluent is a bonus for reuse in that little extra treatment is required for reuse and, therefore, the economics look more advantageous.

Question 13. What percentage of design capacity is in use?

Remarks: As plants near design capacity, decisions on expansion/reuse have to be made. Problems with overloading may be alleviated by reuse.

Question 14. Does the treatment plant (industrial) presently meet discharge requirements?

Remarks: A negative response indicates a wastewater management problem for which reuse may be part of the solution.

Question 15. What quality is the plant effluent in terms of COD (mg/l)?

Remarks: Good quality effluent is a bonus for reuse in that little extra treatment is required for reuse and, therefore, the economics look more advantageous.

Question 16. Are discharge limits set for specific contaminants (i.e., heavy metals, CN, nutrients)?

Remarks: Industrial waste treatment plants with specific contaminant limits are likely to have a higher quality effluent (better for reuse). They may also have problems meeting these discharge standards, in which case reuse may be of assistance.

Question 17. What total quantity of wastewater, treated and untreated, is discharged from the base in MGD?

Remarks: Economies of scale favor reuse at bases with large wastewater volumes.

Question 18. If the base discharges to a municipal or regional sewer system, what is the discharge fee per MG?

Remarks: High discharge fees have a positive effect on reuse economics.

Question 19. Are future changes likely that would markedly increase the discharge fee?

Remarks: Again, future increases in discharge fees can have a positive impact on current reuse planning.

ACTIVITIES: In order to implement a successful wastewater reclamation or reuse system, a large need for subpotable water must exist. These users or reuse sinks for reclaimed water are named activities. In the classic approach, agricultural or recreational irrigation has been a primary sink for reclaimed wastewater. However, at installations with substantial industrial activities that either generate large volumes of reclaimable water or can use reclaimed water as a source of supply, a substantial variation of the classical scenario can be achieved. Questions 20-35 describe the Precursor Model's approach to identification of these activities and their potential impacts as reclaimed water sinks and sources.

Question 20. Does the base have a golf course?

Remarks: Golf courses are the best sink on most bases, as reclaimed water can be used for irrigation. This can be a very high-volume use, depending on the climate.

Question 21. Is the golf course irrigated?

Remarks: If the course is already irrigated, then reclaimed water can be substituted and the existing sprinkler system utilized. However, even if the course is not currently irrigated, if reclaimed water is available, an irrigation system can be constructed.

Question 22. How many acres of landscape and athletic fields could be irrigated if reclaimed water were available?

Remarks: Again, irrigation of landscape and athletic fields is a good sink for reclaimed water.

Question 23. How many large industrial cooling towers does the base have?

Remarks: Cooling tower make-up is another good use for reclaimed water.

Question 24. If the base has a plating shop, how much wastewater does the shop discharge in 1,000 gal/day?

Remarks: Large plating shops provide the potential for water savings by internal treatment/recycle, and usually discharge large volumes of wastewater that, when properly treated, provide a good source of reclaimed water.

Questions 25 through 34.

Remarks: All these questions refer to the presence or absence of key activities on the base that have potential as sinks for the use of reclaimed water. The more important activities are given more value.

Question 35. Does the base have a large pesticide management program?

Remarks: Pesticide disposal can generate highly contaminated wastewaters. Bases with a pesticide management program are rated favorable for reuse, as these toxic contaminants can be removed at the source before they can enter the wastewater system.

INSTITUTIONAL ASPECTS: Experience has indicated that institutional and legal aspects of reuse and reclamation can be critical to the success of proposed programs at military installations. In some instances, legal constraints or negative attitudes towards reuse have stopped programs that were technically feasible and economically justified. Questions 36 through 41 evaluate the institutional considerations of concern at many installations.

Question 36. Is the base free of any long-term water purchase agreements that would prohibit the base from cutting back on water usage?

Remarks: Constraints on the ability to reduce water usage are obviously detrimental to reuse programs.

Question 37. Is the base free of any water laws (i.e., Doctrine of Prior Appropriation) or commitments to regional wastewater systems that would prohibit a reduction in volume of effluent discharged?

Remarks: Constraints on the ability to reduce wastewater discharge volumes are detrimental to reuse programs.

Question 38. Is wastewater reuse occurring now or being planned in surrounding communities?

Remarks: Reuse in surrounding areas portends of local pressures and a favorable legal/institutional climate for reuse.

Question 39. What percentage of the total wastewater generated on-base is currently being reused?

Remarks: Bases already reusing a portion of their wastewater are obvious candidates for a more comprehensive reuse plan.

Question 40. Is there a potential large civilian water user near the base (i.e., golf course, power plant, agriculture)?

Remarks: Large civilian water users near the base can offer a sink for reclaimed water if the quality is sufficient and the economics of transport are feasible.

Question 41. Do key base personnel feel that the effluent is a high-quality source being wasted?

Remarks: The attitude of key personnel towards wastewater reuse is a prime factor in the success of a program.

CLIMATE: Since the value of irrigation, as a major water reclamation sink, is a function of the climatic conditions of the area, the bases located in arid or semi-arid areas have a substantial advantage over installations located in areas of high rainfall. Furthermore, bases that are located in semi-arid or arid regions also are potential candidates for severe potable water shortages, thereby increasing the interest in reduction of total raw water demand. For these reasons, climate is a primary consideration in the initial evaluation of the potential for reclamation at military installations. Questions 42 and 43 are designed to yield preliminary information as to the general climatic conditions of the site.

Question 42. What is the average yearly rainfall on the base in inches/year?

Remarks: Areas with low rainfall are rated more positive for reuse.

Question 43. What is the average yearly evaporation on the base in inches/year?

Remarks: Areas with high evaporative loss are apt to have a higher demand for reclaimed water.

APPLICATION: The Tier I Precursor Model was evaluated at three Army installations: Fort Ord, CA; Fort Jackson, SC; and Anniston Army Depot in Anniston, AL. Table I indicates the results of these trial applications. As indicated by the Table, Fort Ord, CA ranked highest in the reuse category. This would indicate an installation that had substantial potential for water reuse and should be considered for further evaluation in Tier II and Tier III. Fort Jackson, SC scored in the marginal potential category with the strengths of its evaluation for the purposes of reclamation in the water supply and activities categories; however, because of its climate and its current waste disposal procedures, it ranked very low in wastewater, institutional, and climatic categories. For those reasons, it would be likely that benefit could be

obtained from proceeding to Tier II; however, the priority for such an evaluation would be much lower than that of Fort Ord. The last installation, Anniston Army Depot in Alabama, also ranked like Fort Jackson in the marginal reuse potential category. Since Anniston Depot has many large industrial activities and some wastewater management problems, these two categories enhanced the potential for reuse. However, the installation scored very low in areas of water supply, institutional aspects, and climate.

DISCUSSION OF THE TIER II EVALUATION MODEL

The purpose of the Tier II evaluation model is to provide an efficient method for the Army to assess the potential of those installations that score well in Tier I. It should be emphasized that the Tier II evaluation is critical in that the bases that show potential in Tier II are destined for a detailed evaluation and conceptual design at significant expense and investment in time. The Tier II evaluation in itself is a pragmatic engineering evaluation of the installation using a cookbook-style approach. However, due to the more sophisticated nature of Tier II, a substantial amount of engineering judgment in combination with a cookbook-type approach is required to draw sufficiently adequate conclusions.

REQUIRED DATA AND EXPERIENCE: It is estimated that the Tier II type approach will require ten to fifteen man-days for a typical Army installation. The general tasks that will be involved would include three major categories.

1. Base investigation: This section would include a review of existing water and wastewater related records, an interview with key personnel that would facilitate acquisition of base maps, description of activities, existing treatment works (both water and wastewater), their performance and costs, cost of water procurement and treatment if it is externally procured, and future plans and developments. Also included under the base investigation would be some minor sampling activities, of the bucket-stopwatch nature, if required to fill substantial data gaps.
2. Implementation of Tier II evaluation model.
3. Decision as to whether or not to proceed to Tier III type of effort. As indicated in the previous segment, the Tier II type of evaluation requires substantially more data than does the precursor model. On-site surveys, key personnel interviews, and pertinent records must be reviewed. Short summary data needed for Tier II might include things such as activities, their description in terms of daily, monthly water uses and water generated, and seasonal usage patterns; spacial relationships from base maps and exact location of all activities; costs for water supply and wastewater treatment/disposal; types of wastewater treatment and plant performance; future plans for expansion or upgrading of water and/or wastewater systems.

DISCUSSION OF THE MODEL: The Tier II approach is divided into four discrete sections that progress from one to the other. Each section is predicted upon the data generated in the previous section and all ultimately yield a final economic analysis of reuse alternatives. The four sections of evaluation in their order of evaluation include: activities, spacial relationships, conceptual reuse systems, and economics. By evaluating the various components of the models, the activities section, for example, would lead to a complete listing of all installation activities to be considered for inclusion in the reuse systems, while the

spacial relationship evaluation would aid in locating these activities and eliminating those that were far-removed from the main reuse system. The essence of the Tier II model is the third section. Conceptual reuse networks are laid out for eventual cost-comparison and treatment requirements are addressed, including the upgrading of existing facilities. Finally, the most comprehensive scenario of reuse networks that are explored are subjected to a simplified cost analysis. The data generated by the simplified cost analysis then can be used for comparative consideration as to the validity and potential for the installation concerned in terms of its water reclamation reuse potential.

TIER III--WASTEWATER REUSE RECLAMATION POTENTIAL EVALUATION MODEL

The purpose of the Tier III evaluation model is to assist in the selection of the best water reuse systems at installations with high reuse potential. This evaluation is the final phase of the model and will subsequently lead to preliminary engineering designs of the sound reuse networks developed therein. It should be emphasized that Tier III is not to be used for a final design and costing purposes, but rather as a tool to effectively compare the overall estimated cost of selected reuse schemes. The Tier III model is based upon previous investigation by the authors, it is centered upon a complex algorithm that trades off the attributes of the engineering design against the requisite quantity and quality changes within the system. The model requires extensive data in regard to the quality and quantity of the flows at the installation concerned as well as the geographic relationships of the activities to each other. With these data, the evaluation of reuse networks, submitted to the model for processing and the resultant product, is a comprehensive economic evaluation of the reuse network under consideration. Using the output from several potential water reuse networks the engineer can make a judgement as to the most cost-effective reuse network under consideration. This model is not the answer to a problem, but a tool to aid the engineer in the decision-making process.

CONCLUSIONS

1. The water reuse and conservation model adopted during this investigation is a useful planning tool for comparative analysis of potential water reuse systems. The model provides a systematic methodology for organizing the basic information required to evaluate wastewater reuse and conservation potential and analyzing the tabulated data quickly to determine what potential reuse and conservation systems might be feasible. The model requires input and interaction with an engineer experienced in water and sewage management.

Important outputs of the model are the quantity and quality of the supply water required by each activity, quantity and quality of wastewater generated by each activity, treatment required, storage requirements, results of blending of various wastewaters and/or fresh water, pumping, pipeline transport, and estimated system costs.

2. The comprehensive (Tier III) model requires substantial data input describing the individual activities which are potential suppliers or users of reclaimed water. The model is most cost-effective to use when it is anticipated that a significant number of reuse and conservation systems will have to be compared during the planning process in order to select the most cost-effective system or systems. This is facilitated by application of the Tier I precursor model and the Tier II data gathering model to preclude substantial expenditure when, in fact, the Tier III model would not be required due to inadequate need for reuse at the installation.

3. Army installations that use large quantities of water for irrigation and/or industrial activities are the primary candidates for comprehensive water reuse and conservation analysis. These installations include those in areas of below-average rainfall and installations with industrial facilities that must meet stringent effluent criteria.

4. The technology of wastewater reuse and water conservation requires knowledge in three areas: specifically, the pollutant constituents contained in the wastewater generated by the contributing activity or activities. Secondly, the capability of available kinds of waste treatment processes to remove the various pollutants contained in the wastewater generated, and limits of the various pollutant concentrations which can be tolerated by the activities being considered as recipients or users of the treated wastewater. In reviewing the state of the art in these three areas of knowledge, it was found that the first two, i.e., wastewater characteristics and treatment capabilities, are reasonably well-documented or can be determined by sampling, technology transfer techniques, etc. The third area, acceptable pollutant levels for supply water to various activities, was found to be virtually devoid of information. This is understandable, because only isolated, in-process industrial reuse systems are being utilized. As a result of this data shortage, it was necessary, for the purpose of this investigation, to estimate the acceptable water supply quality criteria for various activities. This area is one that could require substantially more research.

5. Results of this investigation indicate that wastewater reuse and conservation can provide a cost savings over fresh water usage at locations with extensive irrigation or industrial water demands, potential water supply programs or high discharge fees. The three-tiered approach to evaluation of these installations for implementation of water reuse, reclamation and conservation systems has proven to be an excellent technique to produce the most cost-effective assessment of the requisite needs of the installation.

TABLE 1. TIER I EVALUATION ON TRIAL APPLICATIONS

Category	Points		
	Fort Ord	Fort Jackson	Anniston Army Depot
I. Water Supply			
1	4	4	0
2	6	0	0
3	9	9	0
4	6	4	4
5	5	5	0
6	3	3	0
7	5	5	2
8	2	4	2
Subtotal	40	34	8
II. Wastewater			
9	7	0	7
10	0	NA	6
11	0	NA	4
12	2	NA	0
13	0	NA	2
14	NA	NA	6
15	NA	NA	0
16	NA	NA	3
17	NA	3	2
18	NA	3	NA
19	NA	0	NA
Subtotal	9	6	30
III. Activities			
20	4	4	0
21	3	3	NA
22	4	4	4
23	0	2	1
24	0	0	4
25	0	0	1
26	2	2	2
27	1	0	1
28	0	0	0
29	2	2	2
30	0	0	0
31	3	0	0
32	1	1	1
33	0	0	0
34	0	2	2
35	0	0	0
Subtotal	20	19	18
IV. Institutional Aspects			
36	4	4	4
37	0	4	4
38	6	0	0
39	0	0	4
40	4	0	0
41	8	0	0
Subtotal	22	8	8
V. Climate			
42	7	0	0
43	4	4	4
Subtotal	11	4	4
TOTAL POINTS	102	72	72

WATER REUSE - THE AIR FORCE EXPERIENCE

Capt Gary R. McNutt
Water Quality Research Engineer
Air Force Engineering Services Center

The Air Force has been conducting research in water reuse since 1973. The purpose here is to share with you some of our experiences during that time in pursuit of the water reuse goal. First "Air Force" must be defined as the Environics Division of the Air Force Engineering and Services Center which was formerly with the Civil and Environmental Engineering Development Office, and before that the Air Force Civil Engineering Center, and initially the Air Forces Weapons Lab. Throughout, Environics has been the lead Air Force lab in environmental quality research and development. Dr Stephen P. Shelton of the University of South Carolina was responsible for much of the effort discussed here during his tour of active duty with the Air Force.

Air Force research and development efforts in water reuse have been directed in three target areas: conservation and recycle within industrial areas; development of a water reuse model to assess the potential for reuse on installations; instrumentation and criteria development in conjunction with the Army and Navy. This paper will deal only with the reuse within industrial areas and the Cascade Water Reuse Model (CWR) because of similar interests in the civilian community.

Preliminary investigations began in 1973 into flange to flange closed loop systems which would use treated municipal wastewater as a source of makeup water for treatment to potable quality. This investigation was a reaction to the anticipated high cost of advanced wastewater treatment necessary to comply with the then newly enacted Federal Water Pollution Control Act Amendments of 1972 and the availability of the resultant quality water resource. It was felt this water was "too good to throw away", especially at bases located in water limited regions.

Advanced treatment processes were reviewed and characterized as part of the initial effort as were the health criteria available at that time. Models were developed of the cost and performance for individual treatment processes and these were incorporated into a computer program for the conceptual design of reuse systems. Some lessons were learned from this effort. First, flange to flange potable reuse was not realistic in 1974. Adequate health criteria did not exist and there was no proven reliability in the advanced treatment processes at that time (1974). This initial effort did strongly suggest two areas for further research, in-plant and in-process recycle of industrial water instead of end of pipe reuse and two, the potential for subpotable reuse and the value of a computer model to evaluate the potential of cascading reuse alternatives.

An example of in-process changes are in electroplating facilities. Wastewater discharged from Air Logistic Centers (ALC) electroplating facilities can account for 50 percent of the flow to that installation's industrial wastewater treatment plant. The electroplating shop is also the largest raw water user at ALC's. Substantial cost savings can be realized if the quantity of raw water required and the volume of wastewater generated by the electroplating facility can be reduced without lowering the quality of the plated product. Rinse tanks in the plating process are a source of large volumes of water contaminated with the plating metals. Investigation showed reverse osmosis could be

applied to chrome plating rinse waters² and is capable of producing a low flow concentrate stream for return to the plating bath to make up evaporation losses and a higher flow product stream suitable for use in air scrubbers.³ Studies showed chromium scrubber efficiency was not strongly influenced by the scrubber water flow rate³ and that chromium concentrations as high as 30,000 mg/l in the recycled scrubber water did not significantly affect air pollution abatement. Thus, a closed loop water reuse operation can reduce raw water needs, recover plating metals, and approach the 1985 zero discharge goal.

Concurrent with the electroplating facility reuse studies, the development of the Cascade Water Reuse (CWR) model was progressing. The CWR model was developed for the Air Force by SCS Engineers as a useful planning tool for comparative analysis of potential subpotable reuse systems at Air Force installations. The model provides a systematic methodology for organizing the basic information required to evaluate the wastewater reuse potential at any base by analyzing tabulated data quickly to determine what potential reuse systems might be feasible. This effort carried forward the earlier modeling as well as characterized water quality and quantity demands of various base activities and defined the extent of quality degradation through use. Typical base activities included waste treatment plants, jet engine test cells, cooling towers, vehicle wash racks, depainting facilities and landscape irrigation.

The model application was conducted in two phases for each of four bases. Phase I was a survey of base activities to include all sources and sinks of water to include quantities generated and discharged and quality requirements and degradation. Phase II was the application of the model. Various networks as suggested by the model were selected and optimized by inputting existing treatment capabilities and "real world" limitations. Costs for the more promising networks were estimated by the model using data for capital costs, operating and maintenance costs, and piping and pumping costs updated to current values as a means of alternative comparison.

Peterson AFB, Colorado Springs CO was selected as having good potential for a subpotable reuse demonstration. Located in semi-arid central Colorado, the base was anticipating problems of water limitations and rising costs. Investigation showed a large portion of the water purchased from the City of Colorado Springs was being used for irrigation of the golf course and base landscape, thus a strong potential for irrigational reuse of treated domestic, industrial and occasional storm water flows. Output from the CWR model was instrumental in the early selection and conceptual design of a reuse system. The project had been carried through to final design when unanticipated obstacles surfaced. Base officials showed a reluctance to accept the risks and uncertainties of a new technology and backed their doubts with legal questions of water rights. The base was buying West Slope water from Colorado Springs and the prior water rights of others downstream were not well defined. Since research funds were supporting this demonstration, time was not available to unravel the legal questions. Instead, negotiations were begun with Colorado Springs to purchase and transport municipal wastewater for base irrigation. Consequently, a new respect was garnered for water law and its impact upon a given reuse scheme.

A comprehensive reuse feasibility study was begun at McClellan AFB, Sacramento, California, following the effort in Colorado.

McClellan AFB had been asked to join a regional wastewater management program. Estimated capital cost to the base to buy capacity in the regional interceptor sewer and treatment plant was \$4 million with additional costs for a trunk sewer, industrial waste pretreatment and future capital costs for tertiary facilities totaling an additional \$3 million. The base fits the criteria for good reuse potential; on-base wells were lowering the water table at least 2 feet per year, 85 percent of nonpotable water use was for cooling towers and irrigation, and the base was faced with high surcharges for discharge to a regional system.

Phase I of the feasibility study was a chemical analysis and flow measurement of all the sources and sinks on this large industrial complex. In Phase II, the CWR model was used to simulate and analyze various alternative system configurations.

After analysis, a recommended reuse system was developed to treat, store, pump, and distribute reclaimed wastewater. Interesting aspects of the system include blending of effluents from the existing domestic sewage treatment plant and industrial waste treatment plant following physical-chemical processes, filtration, and phosphate removal. Ten million gallons of storage are being provided to supply over 10 miles of a dual distribution system. Main uses of the reclaimed water will be landscape and athletic field irrigation, cooling tower makeup water and air pollution control scrubber makeup water. The total construction cost for the system is \$2.5 million, a savings of several million dollars in life cycle costs over the alternative of joining the regional system. An equally important savings is the approximately 400 million gallons annually of fresh water which will not be withdrawn from the groundwater basin. The reuse system should be totally functional in 1979.

The Air Force is currently participating with the Army and the Navy in the development of water reuse criteria for military applications. These applications are unique to the military in field and combat situations. The Army and Navy are now adapting the CWR model to suit their unique requirements as the Air Force continues to assess the potential of bases to use water reuse as an alternate cost effective wastewater treatment method.

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GROUND WATER RECHARGE WITH WASTEWATER HEAVY METAL MOVEMENT

Thomas E. Higgins
Assistant Professor of Engineering
Arizona State University, Tempe

INTRODUCTION

Increasing water usage and pollution of surface water supplies is producing a national water shortage. This shortage is especially acute in the western states, where sources of surface water are expensive to develop or of questionable quality and ground water supplies are being depleted. Locally, the Central Arizona Project (CAP) was initiated to import Colorado River water. CAP water, however, is reported to be similar in quality to that of the treated wastewater in the Phoenix area.¹

Wastewater reuse is imperative. Other potential water sources are of low quality or expensive to transport to the region. A portion of Phoenix's wastewater is already committed to be used as cooling water for the Palo Verde nuclear power plant, presently under construction.

Proposed uses for reclaimed wastewater include irrigation of crops and recharge of ground water for eventual reuse. One potential problem encountered in wastewater reuse is a concentration of toxic heavy metals through each use cycle. Drinking water standards for arsenic, barium, cadmium, chromium, lead, mercury, selenium and silver have been established by the U. S. Environmental Protection Agency.² Additional standards for copper, iron, manganese, and zinc are being established by the Arizona Department of Health.³ These standards are listed in Table 1. Because other potential uses include irrigation and farm animal consumption, recommended limits for these uses are included.

TABLE 1

Water Use Standards (mg/l)

Contaminant	Drinking Water		Irrigation ⁴	Farm ⁴ Animals
	EPA ²	Arizona ³		
Arsenic	0.05	0.05	0.01	0.2
Barium	1.0	1.0	-	-
Cadmium	0.01	0.01	0.01	0.05
Copper	-	1.0	0.2	0.5
Chromium	0.05	0.05	0.1	1.0
Iron	-	2.0	5.0	0.1
Lead	0.05	0.05	5.0	0.1
Manganese	-	0.2	0.2	-
Mercury	0.002	0.002	-	0.01
Nickel	-	-	0.2	-
Selenium	0.01	0.01	0.02	0.05
Silver	0.05	0.05	-	-
Zinc	-	5.0	2.0	25.0

Wastewaters contain varying amounts of toxic heavy metals. These

metals come from corrosion, urban runoff, and from industrial discharges. Conventional wastewater treatment removes a portion of these metals and concentrates them in the form of sludge. Reuse of wastewaters and sludges should be carried out so as to minimize the hazards of these heavy metals.

Soils are seen as a barrier protecting ground water supplies from contamination by pollutants in wastewater. Unfortunately soils may not be a sufficient barrier to prevent contamination of ground water.

Research was initiated to assess specific areas of this potential problem:

1. What are the current heavy metal concentrations in the raw water supplies of the Phoenix area?
2. What concentrations of heavy metals are found in the wastewaters proposed to be reused?
3. What are the forms (soluble, precipitated, or adsorbed) of heavy metals in these sources and how are they affected by treatment processes?
4. What are the mechanisms of heavy metals removal from wastewater passing through soil?
5. What is the long-term capacity of soils to attenuate heavy metal movement and prevent ground water contamination?

The long-term work program of this research is:

1. To survey existing data on heavy metal content of ground and surface waters of the area and to supplement and verify this data by sampling and analysis;
2. To develop a computer model to simulate the effects of chemical precipitation and adsorption equilibria, and ground water movement to predict movement of wastewater containing heavy metals through a soil system;
3. To determine parameters for the model by batch laboratory testing (e.g., adsorption isotherms by multiple jar tests using varying soil and metal concentrations);
4. Prepare pilot columns and determine hydrologic parameters (dispersion) by using a non-adsorbing dye;
5. Run wastewater containing heavy metal through the column to determine overall movement of metals through soils;
6. Use above data to calibrate computer model; and,
7. Use computer model to simulate irrigation and ground water recharge conditions and predict long-term attenuation and removal of heavy metals by soils.

HEAVY METALS AND WASTEWATER TREATMENT

It has been estimated that one municipal usage of water increases the total dissolved solids concentration by 300 mg/l⁵ with part of the increase due to heavy metals.

Wastewaters from urban areas have been found to contain concentrations of heavy metals exceeding drinking water standards.^{6,7} Davis and Jacknow⁸ reported that for three cities (New York, Muncie, Indiana and Pittsburgh) the major heavy metal contamination comes from industry (principally electroplating and photo-engraving) and this is controllable at the source. A significant portion of the heavy metals is contributed by residential usage and this is not so easily managed by source control.

An appreciable portion of heavy metals in domestic wastewater is

removed by conventional treatment processes. Two mechanisms are proposed for the removal of heavy metals during treatment. They are:

1. Precipitation of metal hydroxides and subsequent removal with the sludge; and,
2. Sorption of soluble metals on the sludge.

Precipitation of metal hydroxides is increased with increasing pH and therefore increased removal efficiencies would be expected at higher pH's.

The precipitation of copper, chromium, nickel, and zinc by sewage has been demonstrated by Jenkins *et al.*⁹ Increased removal efficiencies were found for increasing initial concentrations, and increasing pH. Removal resulted in decreased pH due to precipitation of hydroxides.

Investigating toxicity of chromium to the activated sludge process, Moore, *et al.*¹⁰ showed that up to 0.5 ppm of hexavalent chromium was completely removed by the process. McDermott *et al.*¹¹ showed that 50 to 79 per cent copper removal can be effected by activated sludge over a feed ranging from 0.4 to 25 ppm. McDermott *et al.*^{12,13} found removal of zinc to be between 74 to 95 per cent and of nickel to be 30 per cent. Barth *et al.*¹⁴ found that the average efficiencies of activated sludge for removal of hexavalent chromium, copper, nickel and zinc were 44, 75, 28, and 89 per cents respectively.

Argo and Culp¹⁵ showed that tertiary treatment of wastewaters by coagulation, mixed media filtration and carbon absorption were effective at removing cadmium, hexavalent chromium, zinc, and copper and reducing concentrations of other heavy metals. Similar results were found by Linstedt *et al.*¹⁶ Addition of aluminum or calcium salts for phosphate removal also reduced the concentrations of heavy metals.

Because heavy metals are removed in the sludges from wastewater treatment plants they may interfere with the processing of these sludges by anaerobic digestion,¹⁷ and pose a problem in the use of these sludges for agricultural fertilizers.³³

Wastewater renovation is normally required in order for it to meet the appropriate standard for its potential use. Land treatment is proposed as a renovation method.^{18,19} There is the general opinion that passage through soil is a sufficient protection from potential harm resulting from direct wastewater reuse. Contact of wastewater with soil bacteria results in organic decomposition and nitrification. Soils may also afford some protection against metal contamination, although some ground waters have been found to be contaminated by wastewaters.²⁰

Because heavy metals in sludges are concentrated to many times that of the wastewater they came from, the application of wastewater sludges as fertilizers or soil amendments is being carefully researched. It was felt that acid soils would result in the release of metals by dissolution, and thus it was recommended that the use of sludge on acid soils be avoided.²¹ A survey of metal contents of sludges²² revealed high concentrations of cadmium in a commercially available dried sludge fertilizer (Milorganite).

A sample survey was performed of the greater Phoenix metropolitan area water and sewage treatment plants on July 20, 1978. Figures 1 and 2 give a summary of the results for two constituents, lead and chromium.

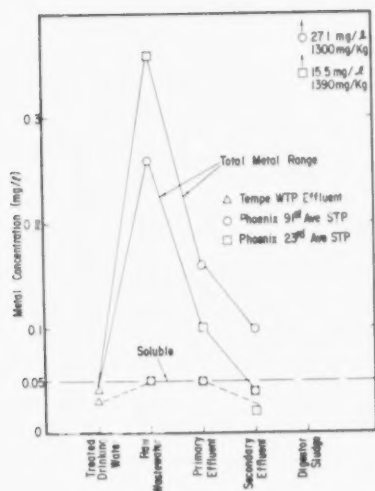


FIGURE 1 Lead Concentrations in Phoenix Water and Wastewater

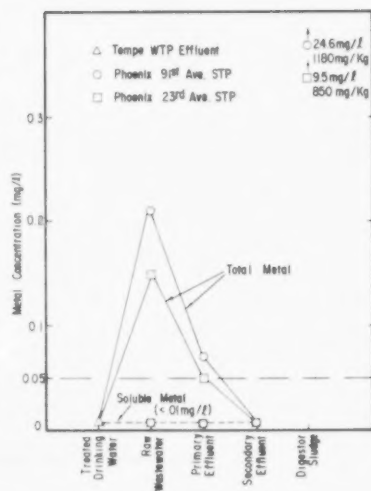


FIGURE 2 Chromium Concentrations in Phoenix Water and Wastewater

In the water supplies, both soluble and total concentrations of these two metals are low (below drinking water standards, as are all constituents tested). The raw wastewaters at both the 23rd Avenue and 91st Avenue sewage treatment plants contain high concentrations of these metals, mostly present in the insoluble form.

These particulate metals are removed by the conventional sedimentation processes of the wastewater treatment plant. The sludges contain very high concentrations of these and of all the metals surveyed. The concentration on a milligram per kilogram dry solids basis shows that the metal influent is greatly concentrated in the sludges.

EFFECTS ON RIVER QUALITY

The Salt and Verde watersheds are the principal water supply for the study area. The water released from the dams on these rivers is diverted for agricultural and urban uses such that the river bed in the Phoenix area is dry most of the year. The only significant source of water flow below the city is from the wastewater treatment plants at 23rd and 91st Avenues. Records of total metal concentrations in the Salt and Verde rivers upstream from the Phoenix metropolitan area and the Gila River below the city have been collected.

A general increase in the concentrations of all metals was noted. Figure 3 shows a mean increase of 900% of chromium. At least a portion of this increase is due to wastewater discharged by the Phoenix area treatment facilities.

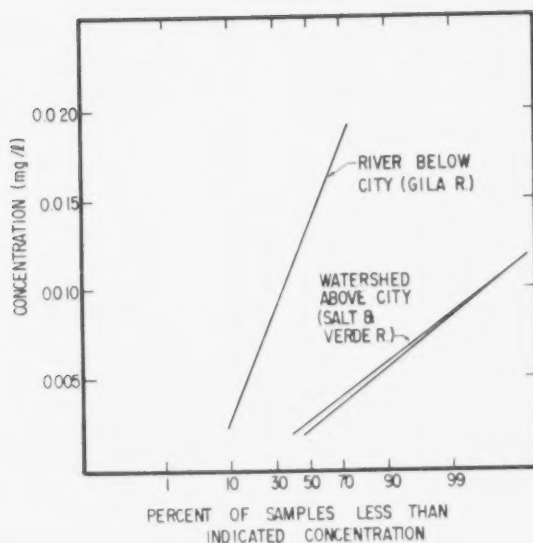


FIGURE 3 Distribution of chemical concentrations (based on the 1968-1969 Phoenix Mountain area, Verde Valley, and Salt Lake Dam) and Gila River (at Salt Lake Dam) for 1964-1970 (Source: Reference 1, 2, 3, 4, and personal communication with Ron Smith, EPA, Phoenix, Arizona.)

EXISTING GROUND WATER CONTAMINATION

Existing drinking water samples from some Arizona wells contain heavy metals in concentrations that exceed State and Federal standards (Table 2). This does not mean that the water supply systems are in violation, but that concentrations of heavy metals in individual wells have been reported to be in excess of the appropriate standards.

TABLE 2

Preliminary Summary of Ground Water Samples Exceeding Drinking Water Standards of Arizona^a

Inorganic Contaminant	Standard ^b mg/l	No. of Samples Exceeding Standard	Max. mg/l	Mean mg/l	S.D.
Arsenic	0.05	31	1.7	0.9	0.31
Cadmium	0.01	3	0.1	0.045	0.047
Chromium	0.05	18	0.20	0.10	0.04
Lead	0.05	11	1.10	0.32	0.37
Mercury	0.005	15	5.0	1.9	1.7

a. Information supplied by the Arizona Department of Health Services, Bureau of Water Quality Control. These are well samples and thus do not indicate violation of Drinking Water Standards in water systems.

b. Minimum values of Federal and State Standards.

HEAVY METALS MOVEMENT IN SOILS

Additional heavy metals removal has been accomplished by percolation of metals containing wastewaters through soil²⁴ suggesting that ground water recharge could result in removal of heavy metals. Clay soils provided the best treatment. The exchanged metal ions were not leached by water.

Movement of heavy metals in coarse soils below sewage disposal ponds was demonstrated by Lund et al.²⁵ indicating that wastewater, injected for reuse should be separated from water supplies sources by fine soils.

Wang and Nacci²⁶ investigated the movement of lead and copper in two Rhode Island soils. Using lysimeters they measured the gross removal of these metals by the soils, not differentiating between sorption and precipitation. Nelson²⁷ reported on studies in Alabama on the movement of metals used in agriculture in soils. Laboratory studies indicated a high degree of soil retention of Ca, Ba, Zn, Pb, Na, Cr, Sr and Mg by the six soils tested. Soluble metal--soil equilibrium concentrations followed Langmuir-Freudlich adsorption isotherms. Mechanisms postulated for metal removal by soils included chelation, surface adsorption, precipitation and physical entrapment.

The proposed mechanism of heavy metal movement in soils is shown in Figure 4.

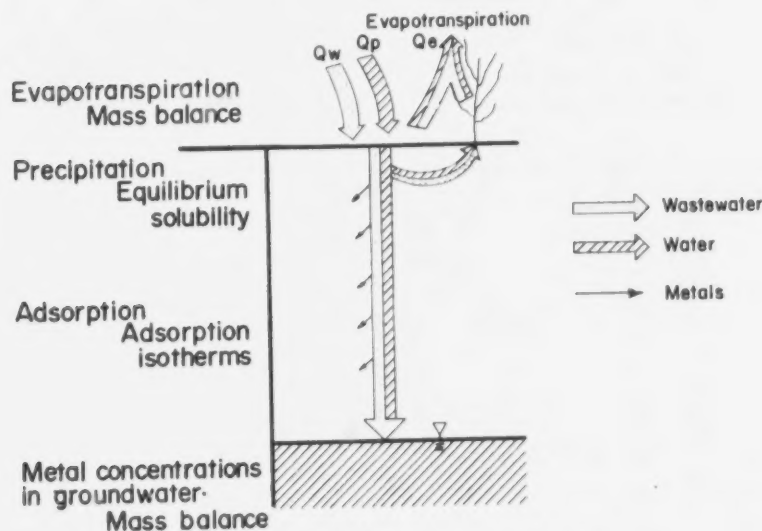


FIGURE 4 Heavy Metal Movement in Soils

The movement of heavy metals consists of the following steps:

1. Wastewater is diluted by rain fall and runoff.
2. A portion of this combined water and wastewater is taken up by plants.
3. A portion of the water is lost to evapotranspiration.

4. Change in concentration may make metals either precipitate or dissolve.
5. A portion of the water percolates through the soil to the ground water.
6. The precipitated metals are filtered out by the soil.
7. Soluble metal concentrations are decreased by adsorption on soils.
8. Dilution occurs by mixing of this water with the ground water, decreasing the metal concentration of the wastewater and increasing metal concentrations in the ground water.

METAL CHEMISTRY

The solubility of metal salts is calculated from their solubility products. Precipitation or solubility of a metal ion (M^{n+}) in equilibrium with a counterion (hydroxide in this case) can be expressed by chemical reaction [1].



The direction and extent of this reaction can be calculated from the product of the molar concentrations of the species involved in the reaction as expressed in equation [2].

$$K_{SP} = [M^{n+}] [OH^-]^n \quad [2]$$

Specific solubility equilibria for metal oxides and hydroxides are shown in Figure 5 as a plot of the maximum concentration of free metal ions vs. pH. Note that solubility of individual metal ions decreases with increasing pH. From this diagram one would expect negligible concentrations of Fe^{3+} , Hg^{2+} , Cr^{3+} and Cu^{2+} at neutral pH. Zn^{2+} , Pb^{2+} , Cd^{2+} , Mn^{2+} , Ag^+ and Ba^{2+} are progressively more soluble with respect to their oxides and hydroxides at neutral pH.

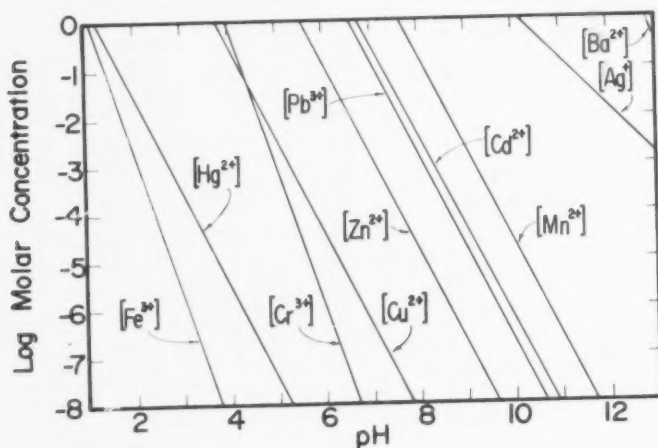


FIGURE 5 Free Metal Ion Concentrations in Equilibrium With Solid Oxides or Hydroxides

In application of sewage sludges to land it is generally stated that to prevent the solution of metals, the pH of the soil should not be acidic.²¹ Table 4 lists the pH values needed to maintain the free metal ion concentration below the Federal and State drinking water standards. Hydroxides and oxide precipitation is not sufficient to protect water supplies from Barium, Cadmium, Lead, Manganese, and Silver.

TABLE 4

pH Below Which Free Ion Will Dissolve to Exceed Standard

Element	Standard mg/l	Molar Conc.	Log Molar	pH
Ba	1.0	7.3×10^{-8}	-5.14	15.4
Cd	0.01	8.8×10^{-8}	-7.05	10.3
Cu	1.0	1.6×10^{-5}	-4.8	6.2
Cr	0.05	9.6×10^{-7}	-6.02	6.0
Fe	2.0	3.6×10^{-5}	-4.45	2.6
Pb	0.05	2.4×10^{-7}	-6.62	9.9
Mn	0.2	3.6×10^{-6}	-5.44	10.37
Hg	0.002	1.0×10^{-8}	-8.00	5.3
Ag	0.05	4.6×10^{-7}	-6.33	16.5
Zn	5.0	7.6×10^{-5}	-4.1	7.6

In addition to free metal forms (eg Cu^{2+}), metals may form soluble complexes with hydroxides and other anions. Figure 6 demonstrates how total metal solubility is modified for Copper (II) in equilibrium with 10^{-2} M total Carbonate. Note that solubility is decreased at low pH's by Malachite and increased at high pH's by the soluble hydroxide complexes.

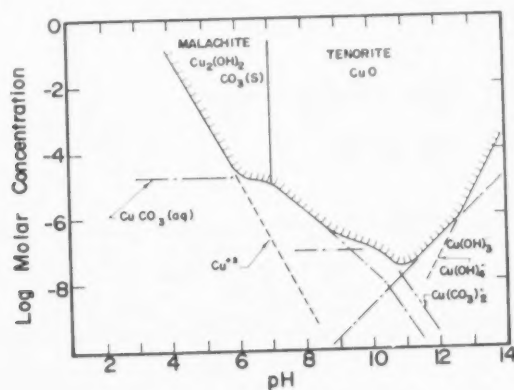


FIGURE 6 Solubility of Copper(II) With 10^{-2} Molar Total Carbonate

Some metals, whose solubility is large with respect to hydroxides or oxides, may still have limited solubility in natural waters due to the formation of salts of other anions. Two examples are lead carbonate and silver chloride. With 10^{-3} Molar total carbonate species, the soluble lead concentration at pH 7 is reduced from approximately 33,000 mg/l to approximately 0.13 mg/l. Likewise with 20 mg/l of Chloride ion present the soluble silver concentration is reduced from many grams per liter to 0.03 mg/l.

ADSORPTION

Heavy metals can further be removed from water by adsorption onto the surface of soil particles. A Langmuir adsorption isotherm describes the equilibrium relationship between the concentrations of ions in solutions and the density of these increases with increased concentration. A typical Langmuir Adsorption isotherm for Cupric ion and Providence Silt is shown as Figure 7.

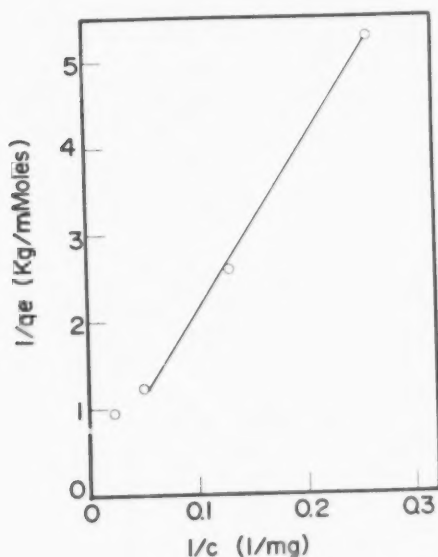


FIGURE 7 Adsorption Isotherm for Copper (II) in Contact With Providence Silt (After Nelson²⁷)

Results reported by James and Healy²⁸ indicate that metal adsorption is not just a reaction of metals to the surfaces of soils, but is a combination of metal oxide or hydroxide formation on surfaces and incorporation of other metals into the oxide. Dissolution of the oxide (eg. by changing the redox potential) would result in the release of the other metals. Jenne²⁹ reported that iron and manganese oxide coatings on soil significantly adsorb other metals.

COMPUTER MODELS

Use of digital computers has made possible the analysis of the complex

systems involved in predicting the movement of heavy metals in soils. Existing computer models such as MINEQL³⁰ and REDEQL2³¹ are dynamic models of equilibrium solubility, capable of predicting precipitation and dissolution of metals with changes in water characteristics. Computer models are also being prepared to describe sorption reactions.³² A model was prepared to show the effects of desorption of Boron and movement in soils.²³ An additional model was reported to consider iron reduction with subsequent dissolution and diffusion through soils.

Heavy metal concentrations in the water at the surface can be determined by mass-balance, taking into consideration flow rates of wastewater, precipitation and evapotranspiration and the concentrations of metal in the wastewater. Equilibrium solubility chemistry can be used to calculate the amount of metal precipitated and filtered in the upper soil and to determine the soluble metal concentration available for adsorption. Downflow through the soil to the ground water table can be treated as flow through a column. The metal concentration of the ground water can be calculated by mass-balance.

The advantage of preparing such a model is that data that can be obtained in a short time on a small sample (eg. adsorption isotherms) can be used to predict the long term effect of the application of wastewaters heavy metals containing to the land. A predictive model of heavy metal-wastewater-soil interactions would be useful in deciding such questions as:

1. Should a particular wastewater be reused?
2. Should heavy metal removal be required of wastewaters in treatment plants prior to reuse?
3. Which soil types are suitable for ground water recharge by wastewaters containing heavy metals?
4. What is the useful life of a ground water recharge area?
5. What concentration of heavy metals is expected in soils irrigated with wastewaters containing heavy metals?

WASTEWATER SOIL INTERACTIONS

Tests were performed on Arizona soils to determine their response (adsorption capacity) to wastewaters containing heavy metals. Cadmium and Nickel were used because of their relative solubility. Portions of treated wastewater (secondary) were contacted with a typical Arizona agricultural soil on a shaker table. The wastewaters were supplemented with acid solutions of nickel and cadmium.

Figures 8a and b show the soluble (filterable through a 0.45 micron cellulose acetate membrane filter) metal concentrations as a function of contact time. Equilibria are established rapidly. The treated wastewater is safely below safe drinking water limits for Cd (dashed line). This soil is quite alkaline (as are most soils from arid regions). High concentrations of these metals are substantially reduced, indicating that a short passage through this soil could considerably reduce even the highest concentrations. Langmuir adsorption isotherms are shown as Figure 9.

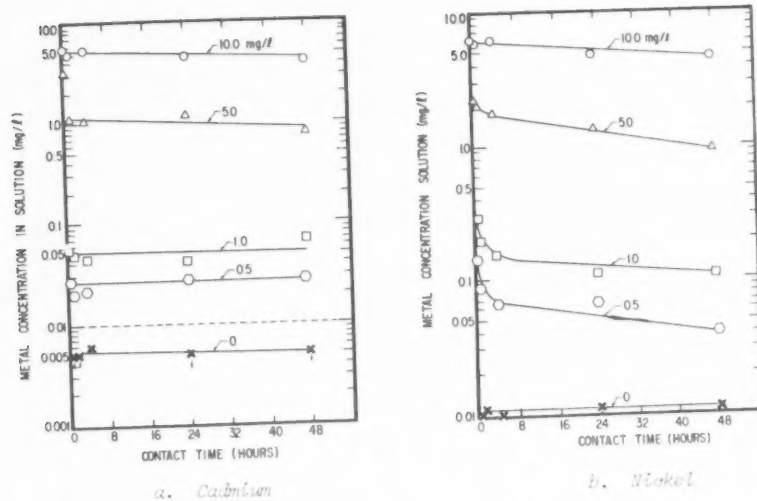


FIGURE 8 Adsorption on Arizona Silty Soil. (200 ml Treated Waste-Water Supplemented With Metal to Concentration Listed/ 2.0 g of soil.)

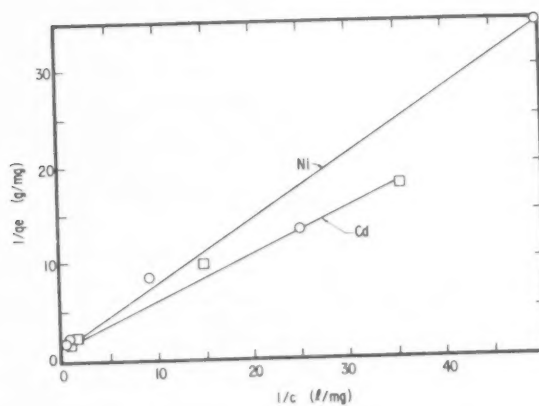


FIGURE 9 Langmuir Adsorption Isotherms for Cd and Ni With Arizona Silty Soil (200 ml Treated Wastewater/2 g of soil.)

CONCLUSIONS

1. The presence of heavy metals may limit the potential reuse of some wastewaters.
2. Conventional wastewater treatment processes can remove a large portion of the heavy metals in raw wastewaters.
3. These metals are concentrated in the sludges, compounding the solids disposal problem.
4. Wastewater discharges in the Phoenix river system increase metal concentrations above background.
5. Some ground waters in Arizona already exceed Safe Drinking Water Standards for metals.
6. Soils may provide physical and psychological barriers helpful in land application of wastewater and ground water recharge.
7. Mechanisms by which soils effect removal of metals include precipitation, filtration and adsorption.
8. It appears that these processes can be formulated into a computer model.
9. Such a model could be used to predict long-term effects of land application of wastewaters containing heavy metals.

ACKNOWLEDGMENTS

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GROUNDWATER RECHARGE WITH WASTEWATER - HEAVY METAL MOVEMENT T. E. Higgins Department of Civil Engineering, Arizona State University, Tempe, Arizona



DEVELOPMENT OF A RAPID COLIFORM DETECTION PROCEDURE SUITABLE
FOR MONITORING RENOVATED WASTE WATER

Anthony M. Cundell - Senior Microbiologist

Janet E. Porter - Research Microbiologist

Anna M. Pisani - Research Microbiologist

Eugene Findl - Director, Bioelectrochemistry
Division

BioResearch, Inc.
Farmingdale, N.Y. 11735

There is a need for a real time coliform detection procedure to monitor coliform bacteria in wastewater being renovated for reuse. Present standard methods of counting coliforms require incubation of inoculated media for at least 24 hours, to allow for colony formation on membrane filters or fermentation of Lactose in broth tubes (American Public Health Association, 1975). Hence, decisions on water quality during the renovation process are, by necessity, delayed.

The criteria for an ideal rapid coliform-counting procedure include:

- a) specificity for fecal coliforms e.g., Escherichia coli,
- b) test completion in less than an hour,
- c) results comparable to those achieved by multiple-tube fermentation and/or membrane filtration techniques,
- d) good reproducibility and accuracy,
- e) ability to detect single coliforms,
- f) amenability to automation,
- g) equipment modestly priced,
- h) procedure should be technically simple, and
- i) potential for field use.

A number of procedures are described in the literature that have potential as rapid coliform-counting techniques (Table I). These procedures are relatively meritorious but are all somewhat less than ideal. For rapidity, the use of an enzyme system (specific to coliform bacteria), as the basis of a counting procedure that permits organism counting without the need for growth, appears to be a sound approach. In this communication the authors wish to report the progress made in the development of a rapid coliform detection procedure based on the hydrolysis of a fluorogenic substrate by the enzyme β -D-galactosidase. This procedure represents a modification of the method used to detect single molecules of β -D-galactosidase in Escherichia coli (Rotman, 1961).

EXPERIMENTAL

The method involves the induction of the enzyme α -D-galactosidase in coliforms using $5 \times 10^{-4}M$ Isopropyl thio α -D-galactopyranoside (IPTG). A cell suspension (5×10^5 to 5×10^8 organisms per ml) of the induced coliform bacteria is dispersed onto a modified microscope slide, containing a layer of silicone oil, using an aerosol sprayer. Within the resultant droplets (5 to 50 μm diameter), a fluorogenic substrate, Fluorescein di (α -D-galactopyranoside) (FDG), is hydrolyzed by the coliform bacteria when the slide is incubated at $35^\circ C$. The FDG concentration was $5 \times 10^{-4}M$. Liberated fluorescein is concentrated within the individual oil-encapsulated droplets enabling them to be counted using a fluorescence microscope (excitation filter BG38, barrier filter yellow 510). A standard curve of the percentage of fluorescent droplets per field of view and the cell density in the coliform suspension was established.

The technique has been successfully applied to count coliform bacteria in primary-treated sewage. (Cundell et. al., 1979). More recently, the rapid coliform detection procedure has been extended from total coliforms to fecal coliforms by elevating the incubation temperature from 35° to $44.5^\circ C$. Using the higher temperature, *Escherichia coli* but not *Enterobacter aerogenes* gave fluorescent droplets using the procedure. Additional work is required to determine whether α -D-galactosidase-positive organisms, outside the classification of coliform bacteria, e.g., *Aeromonas hydrophilia*, *Pseudomonas cepica*, etc., will give false positives in the rapid fecal coliform detection procedure.

With low coliform numbers, a concentration step must be included in the rapid coliform detection procedure. Co-centrifugation of the coliform bacteria with the α -D-galactosidase-negative *Staphylococcus epidermidis* has proved a feasible concentration procedure. Inorganic or organic flocculants may be substituted for the cocci so that large volumes of water samples (up to 5L) may be centrifuged to recover the coliform bacteria.

RESULTS AND DISCUSSION

The percentage of fluorescent droplets per field of illumination, when plotted against the coliform cell density within the range of 10^5 to 10^8 organisms per ml, gave a straight line ($y=36.7 \times -219.8, r=0.95$). To demonstrate the predictiveness of the procedure, a sample of a *E. coli* suspension of unknown cell density was examined. The percentage of fluorescent droplets obtained was 53.9%. This represented a cell density of 2.8×10^7 organisms per ml (plate count 1.8×10^7 organisms per ml).

The authors have confirmed the feasibility of concentrating coliform bacteria by co-centrifugation with excess *Staphylococcus epidermidis* using a fixed angle centrifuge Beckman Model J-21 C, Type JA-20 rotor) (Cundell et. al., 1979). Large volumes (5L) of water samples, containing low cell densities of coliform bacteria, can be concentrated with a continuous flow centrifuge. *E. coli* have a sedimentation

coefficient ($s \times 10^{13}$) of 3,600 (M. Griffith, Beckman Instruments Pers. Com., 1979). Hence, a theoretical flow rate of 200 ml per min. is possible with a rotor speed of 15,000 rpm (12,000 g) during the pelletization of the coliform bacteria (JCF-Z rotor). The bacterial pellet can be resuspended in 5 ml of buffer to obtain a 1,000 fold concentration.

Current work is directed towards automation of the rapid counting procedure. One approach under investigation is the semiautomation of the counting of fluorescent droplets by the addition of a scanning microscope stage and photometer to the fluorescence microscope. The most intense fluorescent droplet in a field of illumination is selected and the excitation light intensity is reduced to limit fading. A suitable photometer sensitivity range is selected to obtain an 80% of full scale detection in response to the fluorescence. To verify that the photometer will not respond to the background, the sight spot (15 μ m, diameter) was focused on the dark background. The scanning stage is moved along, transversing the slide, to avoid counting fluorescent droplets in areas previously illuminated. The number of spots producing responses above a threshold fluorescence intensity are manually recorded from the photometer readout or a graphic record is obtained on a strip-chart recorder. To determine the percentage of fluorescent droplets amongst the total number of droplet, the bacterial suspension can be spiked with a minimal amount of free fluorescein to ensure that all droplets on the modified microscope slide give a response.

Other modifications to the rapid coliform detection procedure that are being investigated include:

- a) the direct counting of the coliform bacteria on membrane filters with a fluorescence microscope operated in an incident mode and
- b) the application of flow fluorometry to the coliform detection procedure.

In light of the frequency of use of coliform determinations in water quality monitoring, a rapid coliform detection procedure would be welcomed by water pollution technologists.

ACKNOWLEDGMENTS

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TABLE 1
EXAMPLES OF RAPID COLIFORM DETECTION PROCEDURES
IN THE RECENT LITERATURE

<u>METHOD</u>	<u>BASIS OF THE PROCEDURE</u>	<u>REFERENCE</u>
Radiometric	Fermentation of C^{14} -labeled lactose to $C^{14}CO_2$	Bachrach & Bachrach, 1974 Reasoner & Geldrich, 1978
Colorimetric	Hydrolysis of onitrophenyl galactopyranoside	Warren, Benoit & Jesse, 1978
Electrochemical (Impedence)	Hydrogen production from lactose	Wilkins, Young & Boykid, 1978
<u>Limulus</u> lysate	Spectrophotometric modification to measure endotoxin	Evans, Schillinger & Stuart, 1978
Chemoluminescent	Bacterial iron porphyrin determination	Picciolo et. al, 1976
Gas Chromatographic	Lactose fermentation-Ethanol detection	Newman & O'Brien 1975
Fluorogenic	Hydrolysis of Fluorescein di (β -D-galactopyranoside)	Cundell, Pisani & Findl 1979
Glutamate decarboxylase	CO_2 production via glutamate decarboxylase	Moran & Wilter, 1976 Leclerc et. al. 1978

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DATA ACQUISITION, MONITOR AND CONTROL SYSTEM FOR A
WATER REUSE PILOT PLANT

Patrick Y. Yang
Program Manager
Life Systems, Inc.
Cleveland, OH 44122

Walter P. Lambert
Sanitary Engineer Staff Officer
U.S. Army Medical Research and Development Command
Fort Detrick, Frederick, MD 21701

Mitchell J. Small
Research Engineer
U.S. Army Medical Bioengineering Research and Development Laboratory
Fort Detrick, Frederick, MD 21701

John R. Gyorki
Department Manager
Life Systems, Inc.
Cleveland, OH 44122

INTRODUCTION

A Data Acquisition and Reduction System (DARS) with control and monitor functions has been developed for a full-scale Water Processing System (WPS) pilot plant. The WPS pilot plant development program had the objective of collecting pilot data for the complete mechanical, hydraulic, chemical and electrical design of a fully automated WPS for the Field Army Medical Facilities as well as other water reuse applications. The DARS with Control/Monitor Instrumentation (C/M I) as its satellites is called the Data Acquisition, Monitor and Control System (DAMCS). To effectively achieve the WPS pilot plant objective, the DAMCS was designed with the following goals: (1) data acquisition of the WPS pilot plant, (2) data storage, retrieval and reduction, (3) pilot plant data report generation, (4) automatically controlling and monitoring the pilot plant, (5) operation by a single operator, (6) operation at the operators station or a remote terminal equipped with a Cathode Ray Tube (CRT) Video Display/Keyboard Unit and (7) computer program development without interruption of the DARS functions.

This system was installed at the U.S. Army Medical Bioengineering Research and Development Laboratory (USAMBRDL), Fort Detrick, Frederick, MD. Figure 1 depicts the DAMCS layout at the installation site. The system consists of foreground and background systems, a built-in monitor and a remote satellite controller with provisions to be expanded to 15 satellite controllers. The foreground system handles real-time tasks which require immediate system attention. The background system performs tasks which do not require real-time service and thus have lower priority in computer task scheduling. Control/monitor and data acquisition are real-time tasks and, therefore, are assigned to the foreground. On the other hand, report generation and data analyses are not real-time tasks and, therefore, are assigned to the background for batch processing. A remote satellite controller is a self-contained, computerized control/monitor instrumentation unit linked to the Data Acquisition, Monitor and Control System via a communication channel.

The background system can be used as a general purpose computer system to develop user-written computer programs in assembly language or FORTRAN. It is also used to retrieve the testing data acquired by the foreground system and generate sensor data reports. Three types of reports have been developed. The on-call report lists operator selected sensor data in engineering units with the date and time of acquisition. The 24-hour report lists the averaged sensor data, high and low peaks of data and the number of excursions beyond sensor setpoints. The command log report lists the operator initiated actions on the satellite controller such as modifications of scale factors, allowable ranges, setpoints and timing sequences and commands to activate/deactivate the water processing system.

The system was designed with communication links to six remote terminals with provisions for immediate expansion to eight terminals. Five of the terminals are within one mile of the mainframe system and were therefore designed with hardwired links. The sixth link is equipped with a modem which can be used for terminals located hundreds of miles away such as the manufacturer's site in Cleveland, OH. These remote links allow users at the remote terminals to operate the Data Acquisition, Monitor and Control System as if the operator were at the mainframe.

BACKGROUND

The mechanical portion of the WPS pilot plant is an integration of several chemical processes such as equalization, prescreening, ultrafiltration, depth filtration, ion exchange, carbon adsorption, reverse osmosis, ultraviolet-activated ozone oxidation and hypochlorination. A more detailed pilot plant description is given in another paper entitled "Multipurpose Water Treatment System Development."⁽¹⁾ The WPS pilot plant was heavily instrumented for process automation, system and personnel protection and scientific data collection. There are 86 electrical actuators and 106 electrical sensors in the WPS pilot plant in addition to mechanical gauges, pressure regulators, valves and other manual adjustments. Among the 106 electrical sensors, 67 are used for process automation and monitoring and 39 are used for actuator status indications.

Figure 2 is the WPS/DAMCS block diagram which shows the system overview. The mechanical portion of the WPS pilot plant consists of three pallets: Water Treatment Unit (WTU), Water Purification Unit (WPU) and UV-activated Ozone Oxidation Unit (O₃/UV). They are designed to process natural fresh, brackish and medical complex wastewater for potable or reuse water production. As shown in the block diagram, there are two types of instrumentation designed for the WPS pilot plant: semiautomatic and automatic C/M I. The semiautomatic and automatic C/M I can be converted back and forth rapidly to allow the project engineers to perform unit process testing under the semiautomatic operation and integrated system testing under automatic C/M I. The automatic C/M I is a functional remote satellite controller of the DAMCS. It is further discussed in another paper entitled "Instrumentation for Automated Water Processing Systems."⁽²⁾ The DAMCS also has a built-in monitor which can be used to bypass the automatic C/M I and acquire sensor data directly from the semiautomatic C/M I.

SYSTEM CONFIGURATION

The DAMCS hardware architecture is a foreground/background distributed processing structure with provisions to accommodate a built-in sensor

data monitor, fifteen external satellite controllers and eight remote user terminals. One satellite controller and six user terminals are implemented in the current design. Its characteristics are shown in Table 1.

One of the DAMCS design requirements was to run data acquisition in the foreground and other lower priority tasks in the background. This foreground/background capability was achieved with a design utilizing multiple Central Processing Units (CPU's) with each CPU designed to control specific tasks. The foreground computer is designed to collect sensor data from the built-in monitor or any of the satellite controllers, the background computer is designed to retrieve sensor data, reduce data and generate reports upon the user's request. When the background computer is not used for report generation, it is available for program development as a general purpose computer. The satellite computers are dedicated to real-time control and monitor functions and to transmit sensor data to the DAMCS foreground computer.

Figure 3 is a functional block diagram of the DAMCS, including its expandability. Figure 4 is the DAMCS hardware block diagram and Figure 5 is a photograph of the DAMCS mainframe.

Foreground System

The heart of the foreground system is a 16-bit minicomputer with 32K words of core memory. Its peripherals include a five-megabyte dual platter moving head cartridge disk and a 23-megabyte, 8 $\frac{1}{2}$ -inch reel, 1600 bits/inch (BPI), Phase Encoded (PE) magnetic tape transport. It has a built-in Analog/Digital (A/D) interface for 160 analog and 128 digital inputs. There is an automatic calling unit which can be used to dial telephone numbers under computer program control. The foreground computer can communicate with one of the six CRT Video Display/Keyboard terminals through the private communication exchange switch. The foreground can also interface with the line printer, card reader, high speed papertape reader and high speed papertape punch through the peripheral sharer. The peripheral sharer is a unique piece of hardware designed for the foreground/background computers to share a common pool of peripheral devices. The connection of the peripheral devices to either computer can be selected by the user at the operator's station by activating a toggle switch.

Background System

The background system has a 16-bit minicomputer with 64K words of core memory. It has a moving head cartridge disk, a magnetic tape transport, a dual floppy disk subsystem, a communication link through the private communication exchange switch to the six CRT Video Display/Keyboard terminals and an interface through the peripheral sharer to the common peripheral devices pool.

The hardware configuration shows that the foreground system has the dedicated hardware for data acquisition functions: the sensor data interface to acquire sensor information directly, the C/M I interface to acquire sensor information from an automatic C/M I and an automatic calling unit to alert an operator upon an emergency. The background computer has more core memory and mass storage capacity. These are needed because of the data retrieval, reduction and program development requirements of the background system.

Computer Links

There are two computer-to-computer links in the DAMCS: One between the foreground and background computers and the other between the foreground computer and the satellite. Electronically these two links are identical. Functionally the link between the foreground computer and the background computer was designed to (1) transmit sensor or system data from the foreground cartridge disk to the foreground computer and then to the background computer during the data retrieval operation and (2) transmit a new program or a new sensor definition file from the background computer to the foreground computer. The link between the foreground and the satellite computers was designed to (1) transmit digitized sensor data from the satellite computer to the foreground computer and then to the cartridge disk or the magnetic tape of the foreground system, (2) transmit operator commands from the foreground computer to the satellite computer and (3) transmit a new program from the background computer to the foreground computer and then to the satellite computer during a new program loading operation.

Remote Terminals

The DAMCS was designed with communication links to six remote terminals with provisions for immediate expansion to eight terminals. Five of the terminals are within a mile of the mainframe system and were therefore designed with hardwired links. The sixth link is equipped with a modem which can be used for terminals located hundreds of miles away.

SOFTWARE ARCHITECTURE

Although the hardware configurations of the foreground and background systems are similar to each other, the software architectures are completely different. The foreground software is a real-time system designed for controlling the tasks of data acquisition, storage and retrieval through the built-in monitor or an external satellite controller. The foreground software was written in assembly language under a Real-Time Executive (RTX) program. The background software is primarily composed of a Disk Operating System (DOS), its utility programs, pilot plant sensor data report generation programs and pilot plant satellite controller loading programs. The report generation and satellite controller loading programs were primarily implemented with FORTRAN IV.

SYSTEM OPERATION

An operator can control the DAMCS from the operator's station or from a remote terminal. The operator can request that the terminal be connected to either the foreground system or the background system. Only two terminals can be connected to the DAMCS at any time -- one to the foreground and the other to the background.

Foreground Operation

The foreground computer is dedicated to DARS functions. The DARS commands are as follows:

- BEGIN - starts the execution of a DARS function
- CANCEL - stops the execution of a DARS function
- DATE - sets or displays the present date

- RATE - sets, modifies or displays the rates at which the DARS collects data for each sensor
- SEND - Transmits a command from the DARS to a satellite controller. This is the mechanism by which the DAMCS operator can control the pilot plant from a remote terminal.
- STATUS - displays the operating condition of the DAMCS
- TIME - sets or displays the present time

The DARS function controlled by the BEGIN/STOP commands are the real-time clock, satellite communication, background communication, sensor data collection and backup magnetic tape storage functions. For example, "BEGIN N-SCAN, S1" command will activate the normal scanning of satellite controller Number One. "BEGIN BACKUP" will start the backup operation whereby data are stored onto the magnetic tape when the moving head disk is full.

Using the SEND command, an operator can transmit a command from the operator station or a remote terminal to a satellite controller. The protocol of the transmitted command would depend on the satellite controller design. In the present system, satellite Number One accepts commands to:

- Select the WPS operating modes
- Request mode transitions including transitions from present mode to Unpowered, Shutdown, Standby, Normal or Auxiliary modes
- Examine parametric data
- Examine or modify setpoints, scale factors or allowable ranges

A proper authorization code is required for an operator to use the SEND command⁽²⁾. The authorization code concepts are described in another paper.

Background Operation

The background computer can be used to retrieve, reduce and report sensor data or to develop new C/M I software. In addition, it can be used as a general purpose computer system using Assembly, FORTRAN IV or BASIC languages.

Satellite Control Program Loader

The C/M I program of a satellite controller can be modified or even redesigned by a user on the background computer. When a new C/M I program is ready to be loaded into the satellite controller the Satellite Control Program Loader can be called by the operator using a standard operating system command. When the Loader is executed the C/M I program is transmitted from the background computer to the foreground computer and then to the designated satellite controller.

After the C/M I program is loaded in a satellite controller, the background computer then queries the user to initialize the sample control and sensor definition programs. If the user decides to do so, the sensor definition file in the background computer is then transmitted to the foreground, replacing the old file.

Report Generation Programs

There are three programs in the background system that produce reports of the testing data stored in the foreground system. The testing data consist of the sensor data and the operator commands recorded on the DAMCS data base for a given satellite. The three report generation programs are:

- The On-Call Report Generator
- The 24-hour Report Generator
- The Command Log Generator

On-Call Report Generator. The On-Call Report Generator lists the individual sensor values recorded for each sensor attached to a given satellite. An example is shown in Figure 6.

The On-Call Report Generator Program was designed to communicate with the operator in an interactive manner. Once the report generator is called by a user the program will begin with a series of questions intended to establish the time period the report should cover. These question and answer interchanges should then establish the beginning and ending point of the samples. Only the samples recorded after the beginning point, but before the ending point, will be included in the report printout. When requesting a report printout the operator may also specify some of the sensors be excluded from the report. In addition, the operator may specify that only one out of a number of records be printed out in the report. For example, the operator may elect to report every fourth record and thus cut down the number of records printed by 75%.

24-Hour Report Generator. The 24-Hour Report Generator collects and summarizes all sensor data stored on the data base over a specified period of time. It does not print out the individual sensor reading as the On-Call Report Generator would. Instead, the average, minimum and maximum sensor data values within the specified beginning/ending time are listed. In addition, the number of excursions beyond limits are included in the report. These numbers are calculated by comparing the sensor data values against the high and low alarm/warning/caution setpoints of that particular sensor. Figure 7 is an example of the 24-hour report.

Command Log Report Generator. The Command Log Report Generator lists all the operator commands entered to a given satellite. Figure 8 is an example of the Command Log Report.

CONCLUSION

The DAMCS provides the user with the benefits of (1) reduced labor requirements in pilot plant operation, (2) reduced labor requirements in pilot plant data acquisition, retrieval, reduction and report generation, (3) increased effectiveness in the pilot plant operation and (4) availability of the system as a general purpose laboratory data acquisition, reduction and computing system.

Although the DAMCS was designed for a specific water processing pilot plant, its DARS portion can be readily shared by a number of different research activities involving sensor data collection, retrieval, reduction and report generation. The distributed design provides the modularity and flexibility that are demanded by the changing and

growing needs under a research and development environment. The interchangeability between the foreground and the background systems also reduced the total system downtime.

ACKNOWLEDGEMENT

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Table 1
DAMCS Characteristics

Parameter	Characteristics
Dimensions (Length x Width x Height), ft	5.5 x 2.6 x 5.7
Weight, lb	<1,600
Power, kW	6.0
Capacity	
a. Sensors, Analog	160
b. Sensors, Digital	512
c. Control Points	384 Digital
Scan Rates, min	5, 15, 30, 60
Scan Rate Selection	Individual
Data Storage Capacity, hr	100 (Worst Case, One Satellite)
Expandability	
a. Sensors, Analog	2,560 by Satellite Expansion
b. Sensors, Digital	8,192 by Satellite Expansion
Operational Requirements	
a. Number of Personnel	1
b. Communication	Interactive
Reporting	24-Hour On-Call Command Log
Formats	Scaled Data and Time High and Low Peaks (24 hr) Excursions Beyond Setpoints Averages (24 hr)
Distances Between DAS and Satellites	Limited Only by Availability of 19.2K Baud Telephone Lines

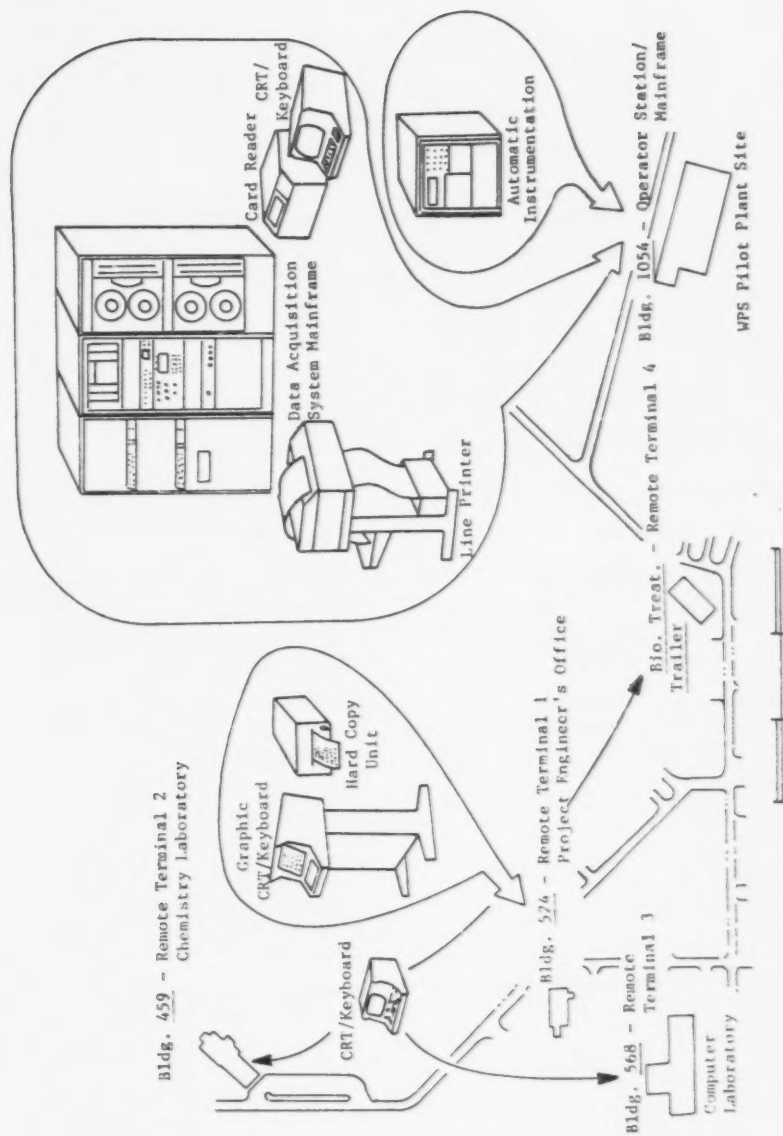


Figure 1 DAMCS Layout

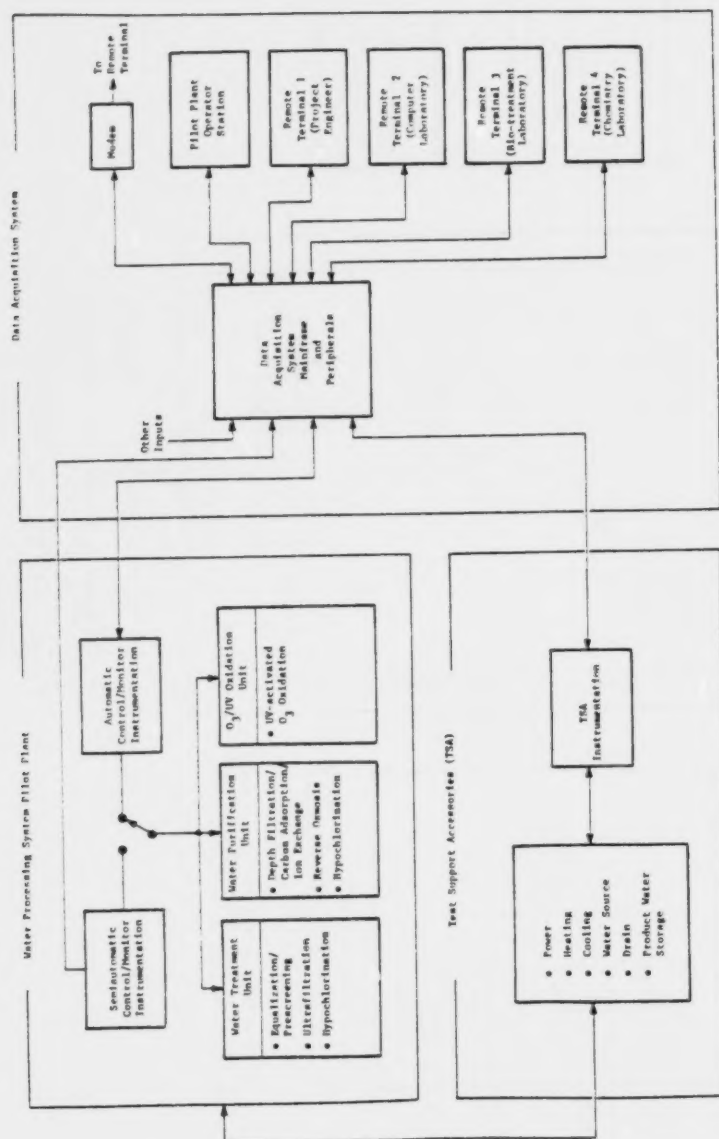


Figure 2 Water Processing/Data Acquisition System Overview

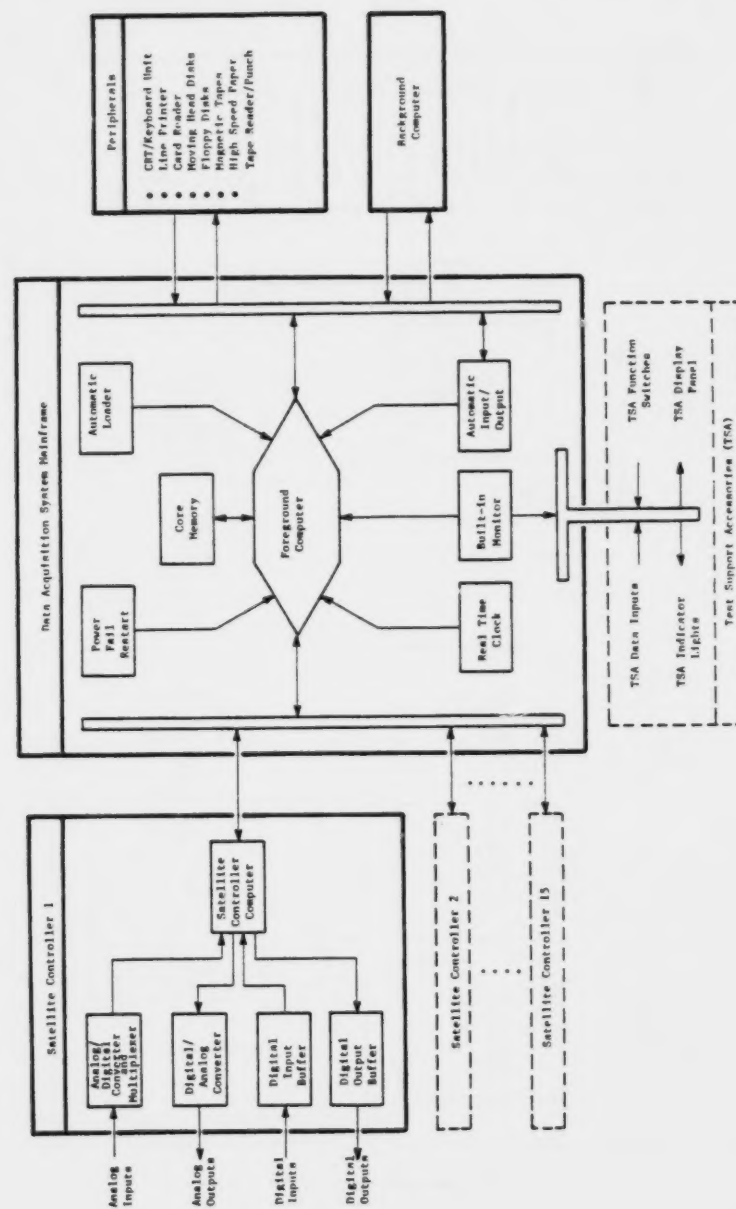


Figure 3 DAMCS Functional Block Diagram

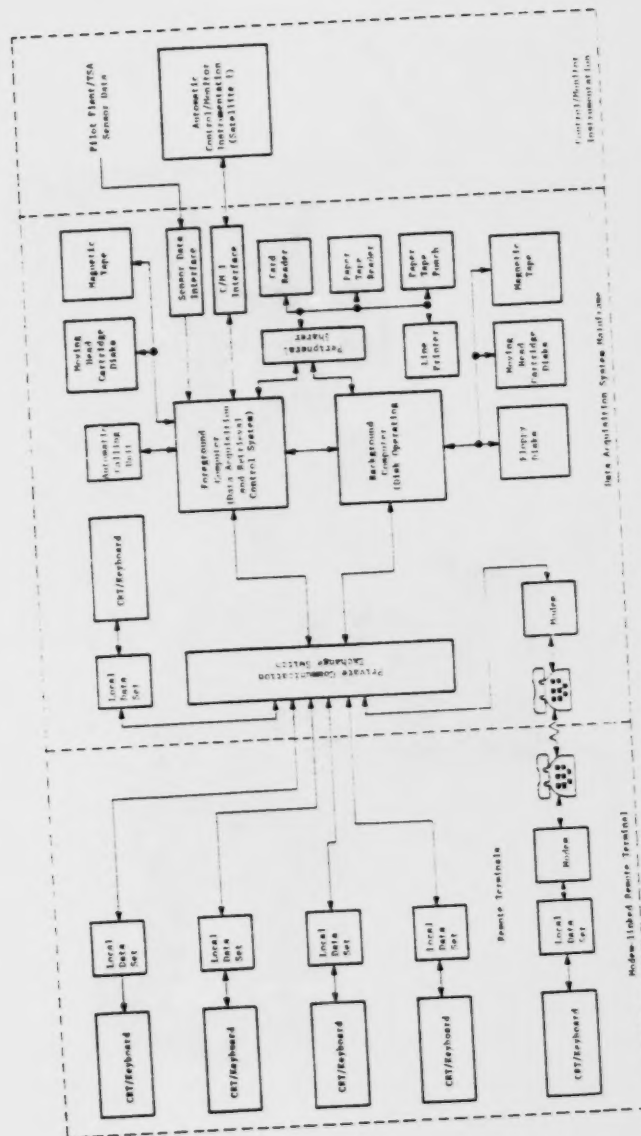


Figure 4 JACS hardware Block Diagram

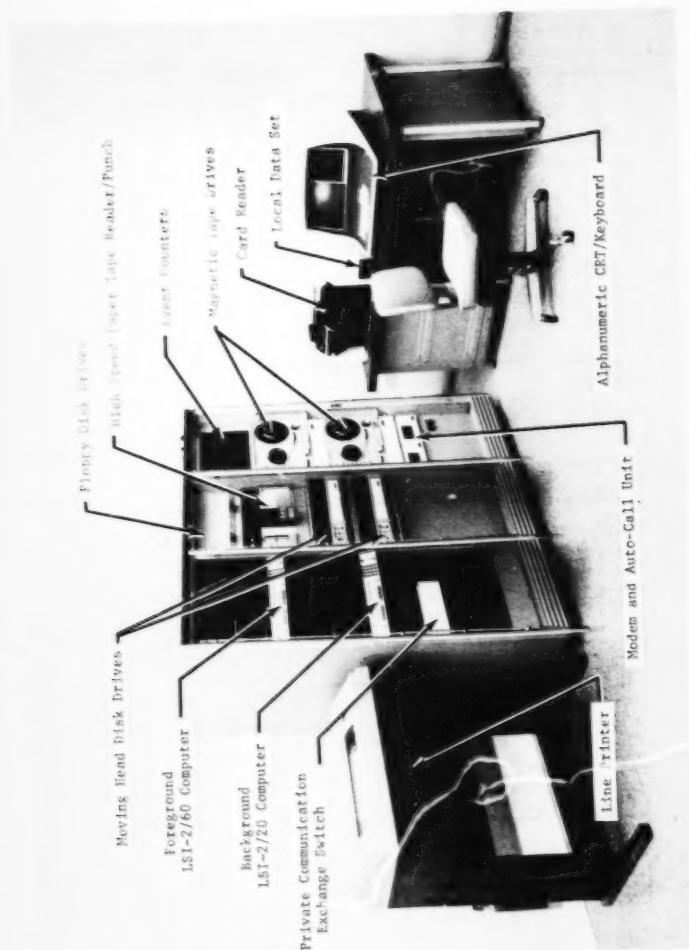


Figure 5 Data Acquisition System

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-----LIFE SYSTEMS, INC. - DATABASE CONTROL SYSTEM-----
CV CALL REPORT          START 10/19/77 00:00    PAGE
SATELLITE 50           STOP 10/20/77 00:00    REPORTED MM/DD/YY 00:01

-----ANALOG SENSORS-----
10/19/77 10:01:20# (IOCM.CODE=VALUE)
20# 205
30# 101
40# 101
50# 101
60# 101
70# 101
80# 101
90# 101
100# 101
110# 101
120# 101
130# 101
140# 101
150# 101
160# 101
170# 101
180# 101
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4090# 101
4100# 101
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Figure 6 On-Call Report

-----LIFE SYSTEMS, INC. - DATABASE CONTROL SYSTEM-----														
24 HOUR REPORT					PAGE									
SATELLITE 50					REPORTED 07/17/78 1711									
START 07/17/78 1200					END 07/17/78 1800									
-----ANALOG SENSOR ANALYSIS-----														
FIGURATIONS BEYOND LIMITS														
CODE	DESCRIPTION	NO.	AVERAGE	UNITS	MINIMUM	MAXIMUM	TIME	SCALE	WARN					
		SERIAL	VALUE		VALUE	VALUE								
1101	TEMPERATURE (CENT.)	1	100.0	°C	100.0	100.0	0000	0000	0000					
1102	TEMPERATURE (F)	2	212.0	°F	212.0	212.0	0000	0000	0000					
1103	TEMPERATURE (C)	3	100.0	°C	100.0	100.0	0000	0000	0000					
1104	TEMPERATURE (F)	4	212.0	°F	212.0	212.0	0000	0000	0000					
1105	TEMPERATURE (C)	5	100.0	°C	100.0	100.0	0000	0000	0000					
1106	TEMPERATURE (F)	6	212.0	°F	212.0	212.0	0000	0000	0000					
1107	TEMPERATURE (C)	7	100.0	°C	100.0	100.0	0000	0000	0000					
1108	TEMPERATURE (F)	8	212.0	°F	212.0	212.0	0000	0000	0000					
1109	TEMPERATURE (C)	9	100.0	°C	100.0	100.0	0000	0000	0000					
1110	TEMPERATURE (F)	10	212.0	°F	212.0	212.0	0000	0000	0000					
1111	TEMPERATURE (C)	11	100.0	°C	100.0	100.0	0000	0000	0000					
1112	TEMPERATURE (F)	12	212.0	°F	212.0	212.0	0000	0000	0000					
1113	TEMPERATURE (C)	13	100.0	°C	100.0	100.0	0000	0000	0000					
1114	TEMPERATURE (F)	14	212.0	°F	212.0	212.0	0000	0000	0000					
1115	TEMPERATURE (C)	15	100.0	°C	100.0	100.0	0000	0000	0000					
1116	TEMPERATURE (F)	16	212.0	°F	212.0	212.0	0000	0000	0000					
1117	TEMPERATURE (C)	17	100.0	°C	100.0	100.0	0000	0000	0000					
1118	TEMPERATURE (F)	18	212.0	°F	212.0	212.0	0000	0000	0000					
1119	TEMPERATURE (C)	19	100.0	°C	100.0	100.0	0000	0000	0000					
1120	TEMPERATURE (F)	20	212.0	°F	212.0	212.0	0000	0000	0000					
1121	TEMPERATURE (C)	21	100.0	°C	100.0	100.0	0000	0000	0000					
1122	TEMPERATURE (F)	22	212.0	°F	212.0	212.0	0000	0000	0000					
1123	TEMPERATURE (C)	23	100.0	°C	100.0	100.0	0000	0000	0000					
1124	TEMPERATURE (F)	24	212.0	°F	212.0	212.0	0000	0000	0000					
1125	TEMPERATURE (C)	25	100.0	°C	100.0	100.0	0000	0000	0000					
1126	TEMPERATURE (F)	26	212.0	°F	212.0	212.0	0000	0000	0000					
1127	TEMPERATURE (C)	27	100.0	°C	100.0	100.0	0000	0000	0000					
1128	TEMPERATURE (F)	28	212.0	°F	212.0	212.0	0000	0000	0000					
1129	TEMPERATURE (C)	29	100.0	°C	100.0	100.0	0000	0000	0000					
1130	TEMPERATURE (F)	30	212.0	°F	212.0	212.0	0000	0000	0000					
1131	TEMPERATURE (C)	31	100.0	°C	100.0	100.0	0000	0000	0000					
1132	TEMPERATURE (F)	32	212.0	°F	212.0	212.0	0000	0000	0000					
1133	TEMPERATURE (C)	33	100.0	°C	100.0	100.0	0000	0000	0000					
1134	TEMPERATURE (F)	34	212.0	°F	212.0	212.0	0000	0000	0000					
1135	TEMPERATURE (C)	35	100.0	°C	100.0	100.0	0000	0000	0000					
1136	TEMPERATURE (F)	36	212.0	°F	212.0	212.0	0000	0000	0000					
1137	TEMPERATURE (C)	37	100.0	°C	100.0	100.0	0000	0000	0000					
1138	TEMPERATURE (F)	38	212.0	°F	212.0	212.0	0000	0000	0000					
1139	TEMPERATURE (C)	39	100.0	°C	100.0	100.0	0000	0000	0000					
1140	TEMPERATURE (F)	40	212.0	°F	212.0	212.0	0000	0000	0000					
1141	TEMPERATURE (C)	41	100.0	°C	100.0	100.0	0000	0000	0000					
1142	TEMPERATURE (F)	42	212.0	°F	212.0	212.0	0000	0000	0000					
1143	TEMPERATURE (C)	43	100.0	°C	100.0	100.0	0000	0000	0000					
1144	TEMPERATURE (F)	44	212.0	°F	212.0	212.0	0000	0000	0000					
1145	TEMPERATURE (C)	45	100.0	°C	100.0	100.0	0000	0000	0000					
1146	TEMPERATURE (F)	46	212.0	°F	212.0	212.0	0000	0000	0000					
1147	TEMPERATURE (C)	47	100.0	°C	100.0	100.0	0000	0000	0000					
1148	TEMPERATURE (F)	48	212.0	°F	212.0	212.0	0000	0000	0000					
1149	TEMPERATURE (C)	49	100.0	°C	100.0	100.0	0000	0000	0000					
1150	TEMPERATURE (F)	50	212.0	°F	212.0	212.0	0000	0000	0000					
1151	TEMPERATURE (C)	51	100.0	°C	100.0	100.0	0000	0000	0000					
1152	TEMPERATURE (F)	52	212.0	°F	212.0	212.0	0000	0000	0000					
1153	TEMPERATURE (C)	53	100.0	°C	100.0	100.0	0000	0000	0000					
1154	TEMPERATURE (F)	54	212.0	°F	212.0	212.0	0000	0000	0000					
1155	TEMPERATURE (C)	55	100.0	°C	100.0	100.0	0000	0000	0000					
1156	TEMPERATURE (F)	56	212.0	°F	212.0	212.0	0000	0000	0000					
1157	TEMPERATURE (C)	57	100.0	°C	100.0	100.0	0000	0000	0000					
1158	TEMPERATURE (F)	58	212.0	°F	212.0	212.0	0000	0000	0000					
1159	TEMPERATURE (C)	59	100.0	°C	100.0	100.0	0000	0000	0000					
1160	TEMPERATURE (F)	60	212.0	°F	212.0	212.0	0000	0000	0000					
1161	TEMPERATURE (C)	61	100.0	°C	100.0	100.0	0000	0000	0000					
1162	TEMPERATURE (F)	62	212.0	°F	212.0	212.0	0000	0000	0000					
1163	TEMPERATURE (C)	63	100.0	°C	100.0	100.0	0000	0000	0000					
1164	TEMPERATURE (F)	64	212.0	°F	212.0	212.0	0000	0000	0000					
1165	TEMPERATURE (C)	65	100.0	°C	100.0	100.0	0000	0000	0000					
1166	TEMPERATURE (F)	66	212.0	°F	212.0	212.0	0000	0000	0000					
1167	TEMPERATURE (C)	67	100.0	°C	100.0	100.0	0000	0000	0000					
1168	TEMPERATURE (F)	68	212.0	°F	212.0	212.0	0000	0000	0000					
1169	TEMPERATURE (C)	69	100.0	°C	100.0	100.0	0000	0000	0000					
1170	TEMPERATURE (F)	70	212.0	°F	212.0	212.0	0000	0000	0000					
1171	TEMPERATURE (C)	71	100.0	°C	100.0	100.0	0000	0000	0000					
1172	TEMPERATURE (F)	72	212.0	°F	212.0	212.0	0000	0000	0000					
1173	TEMPERATURE (C)	73	100.0	°C	100.0	100.0	0000	0000	0000					
1174	TEMPERATURE (F)	74	212.0	°F	212.0	212.0	0000	0000	0000					
1175	TEMPERATURE (C)	75	100.0	°C	100.0	100.0	0000	0000	0000					
1176	TEMPERATURE (F)	76	212.0	°F	212.0	212.0	0000	0000	0000					
1177	TEMPERATURE (C)	77	100.0	°C	100.0	100.0	0000	0000	0000					
1178	TEMPERATURE (F)	78	212.0	°F	212.0	212.0	0000	0000	0000					
1179	TEMPERATURE (C)	79	100.0	°C	100.0	100.0	0000	0000	0000					
1180	TEMPERATURE (F)	80	212.0	°F	212.0	212.0	0000	0000	0000					
1181	TEMPERATURE (C)	81	100.0	°C	100.0	100.0	0000	0000	0000					
1182	TEMPERATURE (F)	82	212.0	°F	212.0	212.0	0000	0000	0000					
1183	TEMPERATURE (C)	83	100.0	°C	100.0	100.0	0000	0000	0000					
1184	TEMPERATURE (F)	84	212.0	°F	212.0	212.0	0000	0000	0000					
1185	TEMPERATURE (C)	85	100.0	°C	100.0	100.0	0000	0000	0000					
1186	TEMPERATURE (F)	86	212.0	°F	212.0	212.0	0000	0000	0000					
1187	TEMPERATURE (C)	87	100.0	°C	100.0	100.0	0000	0000	0000					
1188	TEMPERATURE (F)	88	212.0	°F	212.0	212.0	0000	0000	0000					
1189	TEMPERATURE (C)	89	100.0	°C	100.0	100.0	0000	0000	0000					
1190	TEMPERATURE (F)	90	212.0	°F	212.0	212.0	0000	0000	0000					
1191	TEMPERATURE (C)	91	100.0	°C	100.0	100.0	0000	0000	0000					
1192	TEMPERATURE (F)	92	212.0	°F	212.0	212.0	0000	0000	0000					
1193	TEMPERATURE (C)	93	100.0	°C	100.0	100.0	0000	0000	0000					
1194	TEMPERATURE (F)	94	212.0	°F	212.0	212.0	0000	0000	0000					
1195	TEMPERATURE (C)	95	100.0	°C	100.0	100.0	0000	0000	0000					
1196	TEMPERATURE (F)	96	212.0	°F	212.0	212.0	0000	0000	0000					
1197	TEMPERATURE (C)	97	100.0	°C	100.0	100.0	0000	0000	0000					
1198	TEMPERATURE (F)	98	212.0	°F	212.0	212.0	0000	0000	0000					
1199	TEMPERATURE (C)	99	100.0	°C	100.0	100.0	0000	0000	0000					
1200	TEMPERATURE (F)	100	212.0	°F	212.0	212.0	0000	0000	0000					

Figure 7 24-Hour Report

-----LIFE SYSTEMS, INC. - DATABASE CONTROL SYSTEM-----			
COMMAND LOG	START	06/23/77 06:00	PAGE 1
SATELLITE S1	STOP	06/23/77 18:00	REPORTED MM/DD/YY 00:06

06/23/77	12:40.780		
1234.D	SOURCES D		

06/23/77	12:40.952		
1234.N	NORMAL		

06/23/77	12:41.376		
1234.S	STANDBY		

Figure 8 Command Log Report

RAPID DETECTION OF BACTERIA IN TREATED WASTEWATERS

Randall S. Wentzel
Senior Research Scientist, Atlantic Research Corporation

Patricia E. O'Neill
Research Scientist, Atlantic Research Corporation

Judith F. Kitchens
Chief, Pollution Technology Group, Atlantic Research Corporation

Ralph S. Valentine
Manager, Chemical Technology Department, Atlantic Research Corporation

INTRODUCTION

In water reuse systems, it is important to be able to quickly determine the sanitary quality of the treated water. This need for rapid analysis arises because of limited storage facilities, high demands for the recycled water and the possibility of residual contamination which requires additional treatment. Atlantic Research Corporation has developed a rapid method for the determination of the sanitary quality of water. This method, known as ARCATTM (A Rapid Coliphage Analysis Technique), provides information of the sanitary quality of the recycled water in 4 to 6 hours.

ARCATTM detects bacteriophages in the water sample. Bacteriophages are virus-like particles which invade bacterial cells. Once inside the bacterium, a bacteriophage can utilize the bacterium's metabolic processes to replicate itself several hundred times. Upon completion of replication, the bacterial cell is lysed and the bacteriophages enter the environment. The presence of bacteriophages in a water sample suggests the current or immediate past presence of the corresponding bacteria. The number of bacteriophages enumerated in the ARCATTM test can be quantitatively related to the number of coliform bacteria in the sample. In addition, the presence or absence of bacteriophages in treated wastewater may be used to infer the survival or nonsurvival of viruses through the treatment process.

In this paper, the results of a field test program and in-house research efforts on ARCATTM are reported. The field test program was sponsored jointly by 12 cities throughout the United States, Public Technology, Inc., the National Science Foundation and Atlantic Research Corporation. The cities involved in the field tests were:

Akron, Ohio	Janesville, Wisconsin
Arlington, Texas	Little Rock, Arkansas
Chicago, Illinois	Minneapolis, Minnesota
Clearwater, Florida	Nashville, Tennessee
Dallas, Texas	Philadelphia, Pennsylvania
Fort Lauderdale, Florida	Worcester, Massachusetts

In the field test, the relationships of coliform bacteria to bacteriophages were studied. The effect of water treatment on coliform and bacteriophage survival was also studied. The in-house research concentrated on the effects of heat and pressure on the survival of coliform bacteria and bacteriophages.

BACKGROUND

The survival of bacteriophages in treated waters has been studied by several groups. Wolf and Mixson¹ evaluated the effect of chlorination on coliform and coliphage numbers. These workers found that 99.99% of the total coliforms were removed after 80 minutes exposure to chlorine. However, only 76% of the bacteriophages were inactivated. Kott *et al.*² studied the effect of chlorine dosage and contact time on *E. coli* B, *E. coli* K12, f2 and MS-2 bacteriophages. They found bacteriophage f2 to be the most resistant to chlorine. Survival rates for this bacteriophage were 0.002 to 43% with a chlorine dose of 80 ppm for 2 hours. Kott *et al.*² also measured coliform and coliphage levels before and after chlorination. In this study, 99.9% of the total coliforms but only 90% of the coliphages were removed after chlorination. The presence of suspended particles in the water influences the effect of chlorination on bacteriophages. Freely suspended MS-2 bacteriophage were found to be inactivated approximately twice as fast as clay-attached bacteriophage when treated with hypochlorite.³

Bacteriophages, because of their ease of detection, are being examined for other uses in the water treatment area. Bacteriophage of *Serratia marcescens* have been used as tracers for the reduction of pathogenic bacteria during sewage treatment in South Africa. Kott *et al.*² and Vaughn and Metcalf⁵ have attempted to relate bacteriophages and enteric virus levels in water systems. According to Scarpino⁶, bacteriophages may serve as valid indicators for enteric viruses in some environmental situations such as sewage treatment effluent.

METHODOLOGY

The ARCATTM procedure utilizes a host bacterial culture to challenge the bacteriophages in the sample. The procedure is shown schematically in Figure 1 and described below.

Five ml of an *E. coli* C culture obtained from overnight incubation in Trypticase Soy Broth (TSB) is inoculated into 25 ml of sterile TSB. Upon incubation for 1 hour, a log phase *E. coli* C host culture will be obtained. One ml of the *E. coli* C host culture is added to each of 4 tubes containing 5 ml of sterile melted Trypticase Soy Agar (TSA). Five ml of the water sample are then added to each tube, the contents mixed and poured in a Petri dish. The 4 plates are then incubated at 37°C for 4 to 6 hours. At 4 to 6 hours, the plates are read. Plate reading consists of counting the number of plaques (clear areas in the bacterial lawn). Each plaque is assumed to be made by an initial single bacteriophage.

The ARCATTM procedure described above (or slight modifications to enumerate very low or very high contamination levels) was used by the 12 cities. Each city also performed fecal and total coliform analyses by the standard membrane filter or most probable number procedures. Thus, for a given water sample, the fecal coliform, total coliform and coliphage levels were reported. In addition, water parameters such as turbidity, pH, dissolved oxygen, etc., were included, if available. The coliform bacteria and coliphage levels were transformed to log₁₀ for statistical analysis. A line was fitted to the data using the least squares regression analysis. An analysis of variance table was then used to determine the significance of the line.

The effects of heat treatment were studied using the same methods for detecting coliform bacteria and coliphage levels as described above. The samples were immersed in boiling water under pressure for short periods of time. Total coliform and coliphage levels were determined before and after heat treatment.

RESULTS

The data presented in Figure 2 are the linear relationships between total coliforms versus coliphages and fecal coliforms versus coliphages. The data were collected by 12 cities in 1978 Field Test Program. The statistical analysis of the data is presented in Table 1. Correlation coefficients were 0.62 for total coliforms versus coliphages and 0.68-0.69 for fecal coliforms versus coliphages.

The heat treatment data for coliform bacteria and coliphages are presented in Figure 3. Total coliforms were eliminated from the water sample when subjected to 60°C for 15 seconds. However, coliphages could be detected in water exposed to 90°C for 75 seconds.

The water treatment data, in Table 2, were determined by Ms. Erika Clark at the water treatment plant in Little Rock, Arkansas. Coliform bacteria could not be detected in settled water. Coliphages survived in fairly high numbers in the settled water and could be detected in the majority of filtered and potable water samples.

DISCUSSION

The results of the natural water data indicate a good correlation between coliform bacteria and coliphages. Some improvement in the correlation would be gained if only the membrane filter method of determining coliform bacteria is used.

The effects of heat on total coliform and coliphage levels indicated that coliphages survived to a greater extent. Coliphages would seem to be a more sensitive indicator of contamination in heat-treated water. Also, coliphage absence in a heat-treated water may indicate the absence of enteric virus.

In a water treatment system, total coliform levels were not detectable in the settled water. However, coliphages could be detected through the treatment system. From these data, it appears that coliphages are a more sensitive indicator of the efficiency of water treatment than the total coliform levels now used. A breakdown in the water treatment system can be identified more rapidly with the ARCATTM test than with the currently used methodology. Thus, if the ARCATTM test were routinely employed in water treatment plants, the sanitary engineer would know the quality of the water before it was released for public use. In water reuse systems, knowledge of the sanitary quality of the water is a must before the water is released.

In conclusion, the ARCATTM method offers a new dimension to the rapid determination of the sanitary quality of water. The method yields results in 4-6 hours. It is simple to perform and less expensive than the standard coliform tests currently used. There are significant indications that the bacteriophages are a better indicator of the sanitary quality of water than the current coliform standards. There is also evidence that bacteriophages may be used to infer the presence or absence of viruses in treated wastewater. This ability to infer the presence or absence of viruses is of particular importance in water reuse systems.

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TABLE 1
Statistical Analyses of Natural Waters

Fecal Coliform Levels Versus Coliphage Levels (expressed as log ₁₀)					
Plate Incubation (hrs.)	Number of Samples	Correlation Coefficient	F Value	Log ₁₀ ^b ₀	Log ₁₀ ^b ₁
4	600	.68	524*	.97	.79
6	533	.69	539*	.69	.80
Total Coliform Levels Versus Coliphage Levels (expressed as log ₁₀)					
4	621	.62	397*	2.54	.59
6	611	.62	388*	2.30	.61

*99.9% Significant

TABLE 2
Coliform and Coliphage Survival in Water Treatment

Water Type	Total Coliforms /100 ml	Coliphage /100 ml	Water Treatment
Raw	286	339	-
Settled	0	81	Alum, activated carbon, chlorine gas
Filtered	0	1	Sand and gravel filters
Potable	0	1	Lime and sodium hexametaphosphate

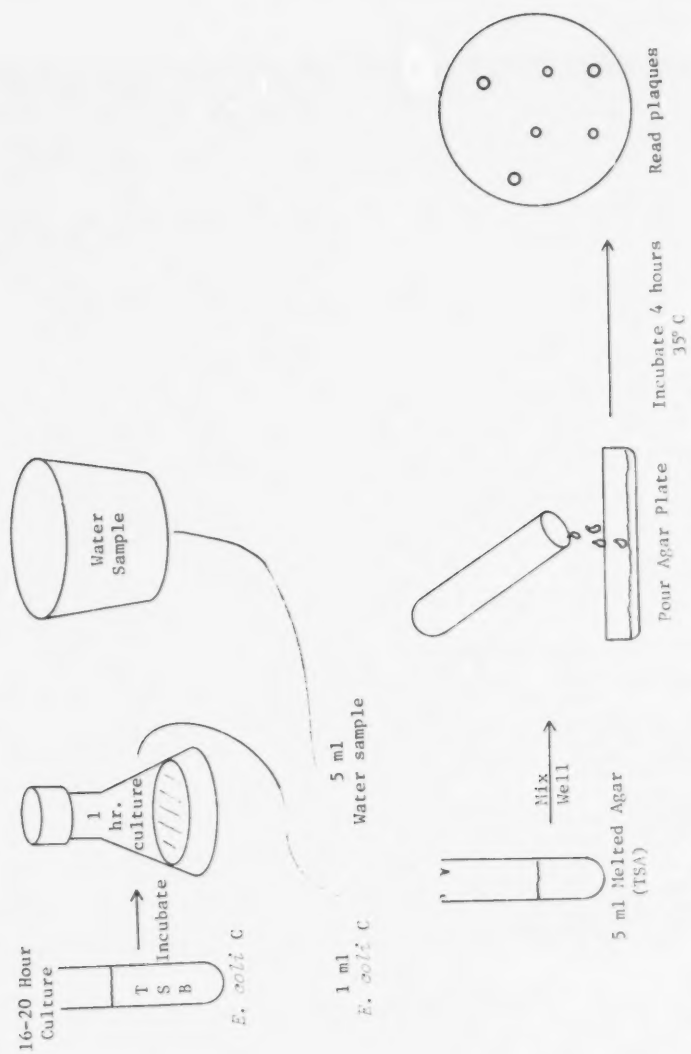


Figure 1. ARC-CAT procedure.

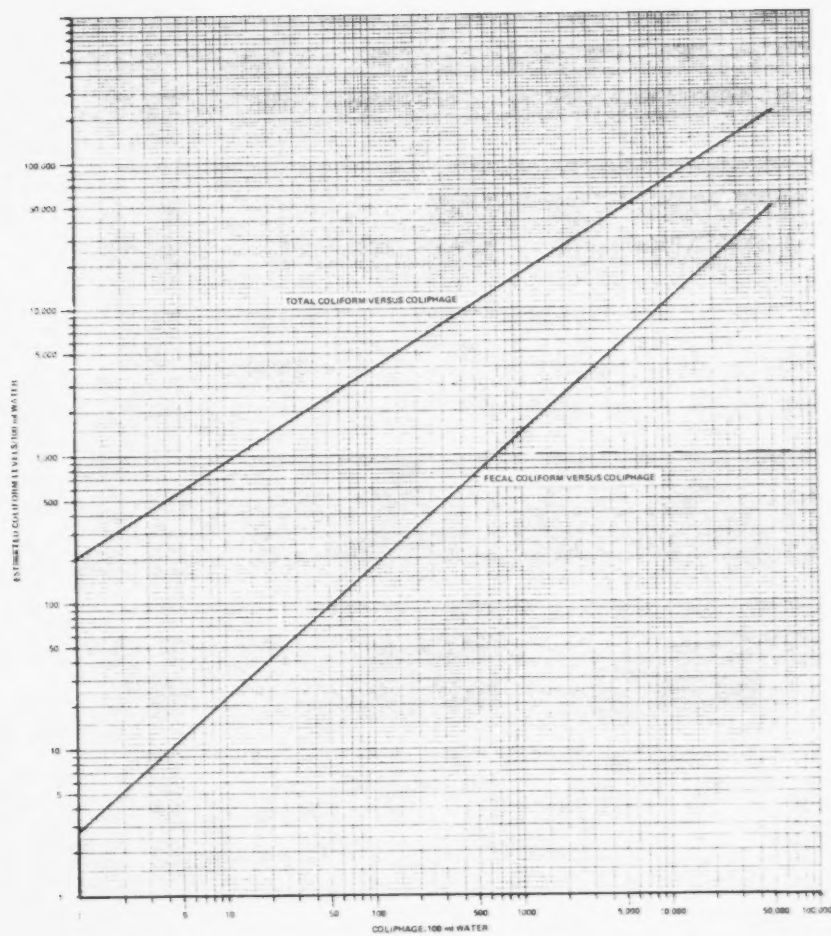
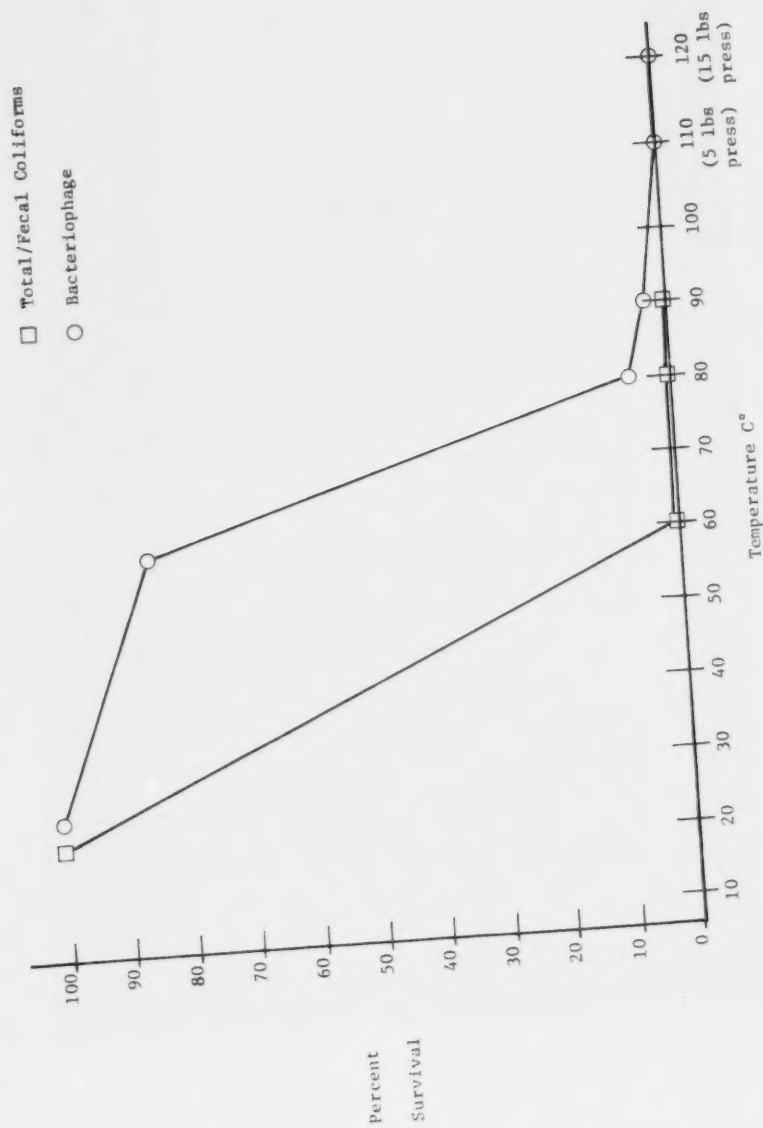


Figure 2. COLIFORM-COLIPHAGE RELATIONSHIP FROM 1978 DATA



RAPID DETECTION OF BACTERIA IN TREATED WASTE WATERS

J. F. KITCHENS R. S. WENTSEL

P. O. NEILL A. S. VALENTINE

OBJECTIVE

Rapid determination of water quality in waste water treatment plants.

METHOD

The method was based on the use of a rapid detection system. The system was designed to detect the presence of bacteria in waste water. The system was designed to detect the presence of bacteria in waste water.



RESULTS

The results of the rapid detection system are shown in the following table.

Waste Water	Rapid Detection System	Rapid Detection System
100	100	100
200	200	200
300	300	300
400	400	400
500	500	500
600	600	600
700	700	700
800	800	800
900	900	900
1000	1000	1000

The results of the rapid detection system are shown in the following table.

Waste Water	Rapid Detection System	Rapid Detection System
100	100	100
200	200	200
300	300	300
400	400	400
500	500	500
600	600	600
700	700	700
800	800	800
900	900	900
1000	1000	1000

ATLANTIC RESEARCH CORPORATION

1000 ATLANTIC AVENUE
NEW YORK, N. Y.

CONCLUSIONS

The results of the rapid detection system are shown in the following table.

Waste Water	Rapid Detection System	Rapid Detection System
100	100	100
200	200	200
300	300	300
400	400	400
500	500	500
600	600	600
700	700	700
800	800	800
900	900	900
1000	1000	1000

The results of the rapid detection system are shown in the following table.

Waste Water	Rapid Detection System	Rapid Detection System
100	100	100
200	200	200
300	300	300
400	400	400
500	500	500
600	600	600
700	700	700
800	800	800
900	900	900
1000	1000	1000

VIRUS CONCENTRATION FROM WATERS BY HOLLOW FIBER ULTRAFILTRATION:
THE IMPORTANCE OF HYDRODYNAMICS

Georges Belfort*, Yael Rotem-Borensztajn, Elie Katzenelson
Environmental Health Laboratory
Hebrew University-Hadassah Medical School
Jerusalem, Israel

ABSTRACT

The performances of poliovirus 1 concentration from tap water using three different hollow fiber ultrafiltration modules are compared. Analysis of the results in terms of the virus/fluid dynamics in the hollow fibers is presented.

The virus concentration results are analyzed for laminar flow in a circular porous tube using a temperature compensated modified filtration equation to describe permeation flux; solute (poliovirus 1, turbidity and dissolved organic carbon) removal is characterized by retention efficiency and an empirical hold-up index; and dispersion and particle capture are considered under conditions where permeation forces are usually far greater than forces due to Brownian motion.

The use of these hydrodynamic models permit the comparison of different modular designs on a normalized basis, provide the formalism with which to analyze solute concentration by hollow fiber ultrafiltration, and allow a priori evaluation of a untried candidate module for such applications.

A major result of this study is that the performance of all the modules are limited by their active surface area. Modules with larger surface areas would drastically reduce the duration of the process without significantly effecting recovery efficiency.

INTRODUCTION

The need to develop reliable and simple analytical methods to concentrate viruses from large volumes of different quality waters is intensifying as direct and indirect reuse of wastewater for municipal consumption continues to increase. This is predicated by the fact that dangerous human pathogens including viruses are possibly present at very low concentrations in some drinking water sources(1).

Several methods, such as filter adsorption-elution procedures (2,3), precipitation by or adsorption to various charged species (4,7), polymer two-phase separation (8,10), membrane filtration methods (11) (including ultrafiltration, reverse osmosis, hydroextraction, and electro-osmosis), and other methods (11) (including centrifugation, electro-phoresis, freeze concentration, gauze pad and affinity chromatography), have all been suggested and studied as a first step to concentrate model and other viruses from large volumes of water for enumeration purposes. Of these, only two methods meet the large initial volume, low initial virus concentration, and reliability criteria usually listed as necessary for the development of a successful standardized method (11). These are the two step continuous filter adsorption-elution procedure and the tangential flow ultrafiltration method.

*Direct inquiries to: Dept. of Chem. & Env. Eng., Rensselaer Polytechnic Institute, Troy, NY 12181

The ultrafiltration method is slower than the adsorption-elution method, but it is simpler, results in a higher first step virus recovery efficiency, and may be more economical with, ultrafiltration modules that are sterilizable and reuseable. A recent comprehensive evaluation of the two-step microporous filter adsorption-elution method, which was recently introduced as a tentative standard method for enteric virus concentration from tapwater (13), has concluded that the method has serious deficiencies with generally poor virus recoveries (14). Both methods are most suitable for relatively clean waters containing low concentration of competitive adsorbers and clogging compounds. Little attention has been given to the use of tangential flow ultrafiltration for virus concentration with sewage or treated effluents, especially at high Reynold's numbers. Efficient and reliable concentration methods for detecting small quantities of viruses in large volumes of turbid natural waters such as river, lake and coastal waters are not yet available.

We have devoted our attention to hollow fibers or capillary ultrafiltration membrane methods because of their compact size (high surface area/unit volume), well-controlled hydrodynamics (for bore-feed flow), commercial availability with different membrane materials and molecular weight cut-offs, sterilizability, and finally, their ability to concentrate large volumes (up to 100 l so far) of model poliovirus 1 from deionized and tap water in short periods (1 to 2 hours) without allowing the virus to permeate the asymmetric hollow fiber membrane (15-17).

The purpose of this paper is to compare and interpret the performance of three different hollow fiber-capillary membrane modules in concentrating poliovirus 1 from deionized and Jerusalem tap water with respect to their hydrodynamics. Results for the first two hollow-fiber modules have been published in earlier papers (15,16) while the most recent results for the capillary module has recently appeared in the literature (17). It is also the intention here to show that by understanding and controlling the fluid mechanics of the modular system virus concentration performance can be optimized.

THEORETICAL

Modified Filtration Equation

When comparing the performance of different hollow fiber modules or different runs for the same module, it is preferable to analyze the results as a function of reduced accumulated permeation volume, $\bar{V}_p$. Belfort and Marx (18) have adapted the standard filtration approach to study fouling of reverse osmosis membranes. Their *modified filtration theory* will be described below for analysis of the hollow fiber ultrafiltration experiments reported later.

The modified-filtration equation ignoring osmotic effects ($\Delta\pi \approx 0$) and imperfections in the membrane such as micro-pin-holes, can be written as follows:

$$\frac{1}{\bar{V}_p} = \frac{\mu \bar{c} D_w}{R \Delta P_c S} + \frac{\mu r' D_w}{\Delta P_m R} \quad (1)$$

where the total accumulated weight of suspended solids deposited on the membrane surface is given by

$$W = \int_0^{\bar{V}_p} w \, d\bar{V}_p \quad (2)$$

For constant suspended solids concentration w_o in the feed such as in cake filtration or for ultrafiltration with no feed recycle

$$W = w_o V_p \quad (3)$$

On substituting Eq (3) into Eq (1), the following results

$$\frac{-1}{V_w} = k_1 V_p + k_2 \quad (4)$$

where

$$k_1 = \frac{u \alpha D_w V_{fi} w_o}{R \Delta P_c S}; \quad k_2 = \frac{u r' D_w}{\Delta P_m R}$$

For batch recycle of the feed in an ultrafiltration system, such as used in these studies, the predominant force acting on the suspended particles is due to permeation (see later discussion comparing permeation with Brownian motion). Experimentally it has been found that the solute feed concentration remains relatively constant during the dehydration process (i.e. $w = w_o$). Thus Eqs (3) and (4) are also valid for this case.

Solute Removal

As described previously (15), a convenient method to compare the removal rates of solutes (number of particles N removed) and fluid (volume V_f removed) from the feed solution during an ultrafiltration experiment uses the following equations:

$$\bar{N} = (\bar{V}_f)^{1-R} \quad (5)$$

or

$$\bar{C} = (\bar{V}_f)^{-R} \quad (6)$$

The reduced number $\bar{N}$ or reduced concentration $\bar{C}$ represent quality parameters such as virus number (PFU), turbidity (FTU) or dissolved organic carbon (DOC in mg/l). From a plot of $\log \bar{N}$ (or $\log \bar{C}$) versus $\log \bar{V}_f$, the retention efficiency or average rejection R is obtained. $R \rightarrow 1$ implies $\bar{N} \rightarrow 1$ or $N_f \rightarrow N_{fi}$ which means there is no loss of N_f during the experiment and $\bar{C} \rightarrow \bar{V}_f^{-1}$. $R \rightarrow 0$, on the other hand, implies that N_f is lost from the solution at approximately the same rate as $\bar{V}_f$ is reduced resulting in $\bar{C} \rightarrow 1$ or a constant concentration during the dehydration experiment.

These conclusions can best be explained from the usual definition of retention, viz.

$$R = 1 - v_s / (v_w C_f) \quad (7)$$

Thus for $R = 0$, the solute flux $v_s = v_w C_f$ and is simply due to permeation. For $R = 1$, the solute flux $v_s = 0$ since v_w and C_f are observable non-zero quantities here.

During the experiment the solute could either be adsorbed onto or into the membrane or it could pass through the membrane with the permeate. With the assymetric membranes of low molecular weight cut off used in this study, the viruses apparently do not permeate the membrane but are adsorbed for later recovery by backwashing (16,17).

Convective-Induced Polarization and Particle Capture

When a stationary or moving fluid containing a solute is exposed to

some external field, the possibility exists that the fluid and/or the solute will experience motion in the field direction. Examples using this concept for the removal or separation of either dissolved or particulate solutes from gaseous and liquid phases abound in the literature. "Polarization Chromatography" (PC) is an example of this and is a relatively new process which uses different transverse polarizing potentials or driving forces to move dissolved solutes out of a slow moving carrier fluid (19,20). Convective-induced polarization has been successfully used to fractionate proteins (21), viruses (22), and polystyrene beads (23). A schematic of the lateral concentration distribution of the retarded solute under convective-induced polarization is shown in Figure 1.

A rigorous exact analysis using the generalized dispersion theory to solve the model equations for PC has appeared in the literature (24). Of special interest here, is the retardation factor for a tubular hollow fiber ultrafilter (25).

$$R_s \equiv u_s / \langle v_z \rangle = \frac{4}{\sqrt{\pi P\delta}} \frac{(e^{-Pe/4} - e^{-Pe})}{(\text{erf } \sqrt{P\delta} - \text{erf } \sqrt{P\delta}/4)} \quad (8)$$

For sufficiently high $P\delta$, Eq (8) reduces to

$$P\delta \lim_{P\delta \rightarrow \infty} (R_s) = 4/P\delta \quad (9)$$

In the above analysis, it has been implicitly assumed that the solute, s has a center of mass axial velocity, u_s . In some cases with very high $P\delta$, the dispersion forces at the membrane-solution interface may overcome the repulsion forces and result in an essentially immobilized solute ($u_s \approx 0$). The capture of particles from flowing fluids has important applied significance such as aerosol filtration, gas cleaning and particle capture in the bronchial tubes. Trajectory calculations to determine the rate of depth filtration (particle deposition) have also been developed (26,27). Recently Hung and Tien (28) published a theoretical analysis of the deposition of particles in porous reverse osmosis tubes with very low seepage rates.

Assuming particle attachment on contact with the membrane surface and negligible re-entrainment (29), the entrance coordinate of the limiting trajectory, $R_1(z=0)$ shown in Fig. 1 can be obtained from

$$R_1(0) = R - v_w L / \langle v_z \rangle \quad (10)$$

Assuming an exponential trajectory for simplification (Hung and Tien's solution should be used, however for a qualitative estimate, Eq (11) will suffice here)

$$R_1(z) = R_1(0) \exp(bz) \quad (11)$$

where an average $\langle R_1(z) \rangle$ is defined by

$$\langle R_1(L) \rangle = \frac{1}{L} \int_0^L R_1(z) dz \quad (12)$$

The average velocity of the fluid in the escape region is given by

$$\langle v_z \rangle_{es} = \frac{\int_0^{2\pi} \int_0^L \langle R_1(L) \rangle v_z r dr d\theta}{\int_0^{2\pi} \int_0^L \langle R_1(L) \rangle r dr d\theta} \quad (13)$$

where the velocity distribution in a tube with low seepage rates is given by

$$v_z = v_z^0 \left[1 - \left(\frac{r}{R} \right)^2 \right] \quad (14)$$

Substituting Eq (14) into (13), the average escape velocity

$$\langle v_z \rangle_{es} = 2v_z^0 \left[0.5 - 0.25 \left(\frac{\langle R_1(z) \rangle^2}{R} \right) \right] \quad (15)$$

Taylor Dispersion Analysis (30)

Under conditions of low seepage rates (i.e. such that Eq (14) holds) and using a conservative tracer (poliovirus 1), Taylor's dispersion analysis for laminar flow in pipes could be used to determine the diffusivity D of poliovirus 1. Using the formalism outlined in Levenspiel's text (31) to describe the residence time distribution (RTD) resulting from an impulse, the intensity of fluid mixing and the experimental retardation factor [see Eq (8)] could be determined directly.

From a plot of reduced concentration E versus reduced time θ , the theoretical variance σ_θ^2 is matched to the response data for a closed vessel such that

$$\sigma_\theta^2 = 2\alpha - 2\alpha^2(1 - e^{-\alpha})^{-1} \quad (16)$$

where $\alpha = D/(\langle v_z \rangle L)$ is the reactor dispersion number and is obtained by trial and error. D , the dispersion coefficient is then used to calculate the diffusivity D from

$$D = \frac{(Rv_z^0)^2}{192 D} \quad L \gg 3R \quad (17)$$

under conditions of high flow rate where dispersion is caused by convection rather than by diffusion.

Permeation versus Brownian Motion

Since all the concentration experiments were conducted with the hollow fibers aligned vertically, the gravity force is neglected as it acted along the axis of the fibers and played no role with respect to particle capture at the membrane-solution interface.

The relative importance of Brownian motion with respect to permeation however must be evaluated. Following Hung and Tien's (28) approach, and assuming that the particle concentration over a given cross-section is approximately uniform ($C_b = C_f$), the amount of particle deposition by Brownian Motion is given by (32)

$$v_B = k_B C_b \quad (18)$$

where

$$k_B = 0.746 \left(\frac{D}{R} \right) \left(\frac{4 \langle v_z \rangle R^2}{D_p} \right) 0.338 \quad (19)$$

$$D_p = kT/(6\pi\eta a) \quad (20)$$

v_B should be compared with the particle deposition simply due to radial permeation,

$$v_S = v_w C_f \quad (21)$$

Thus the magnitudes of k_B and $|V_w|$ should be compared for a range of typical particle sizes, a , for a range of hollow fiber inside radii R , and for different path lengths z . This comparison is presented in Figs. 2 and 3 for the three modules tested.

Brownian diffusion is less important than permeation for particle deposition for both the Berghof (BMR 500515) and the Amicon (HIDPIO) modules. This appears to hold for the whole range of particles and for the entire path length of both modules. The Dow Miniplant 80, however, behaves differently. Only for particle diameters greater than 8.73×10^{-6} cm is $|V_w| > k_B$ for all $z > 0.05 L$.

For enteroviruses such as poliovirus ($a = 1.5 \times 10^{-6}$ cm), coxsackievirus ($a = 1.5 \times 10^{-6}$ cm) and echovirus ($a = 10^{-6}$ cm) it can be seen in Figure 2 that for about 50% of the path length Brownian diffusion dominates in the Miniplant 80. For the other two modules, however, permeation forces dominate throughout the path length. Although clearly ideal, this analysis can be used qualitatively to (1) compare different modules, (2) assist in designing an optimum hollow fiber system for a particular application, and (3) explain experimental results such as high solute hold-up during concentration (see later discussion).

Solute Hold-up Index, $\bar{H}$

As described in the previous section, under certain conditions the permeation force is the dominant force acting on the fluid and solute particles during their passage through the hollow fiber module. For a given module geometry (R, L, n) permeation characteristics (v_w) and average recycle velocity inside the tubes ($\langle v_z \rangle$), it would be convenient to have a measure of the potential solute hold-up or retention during a concentration run. $(1-R)$ is just such a measurement. Unfortunately, R is difficult and expensive to obtain in the field and although it is easily measured in the laboratory it requires a costly and time consuming effort to characterize a particular module and operating conditions.

A simple, easy to get, qualitative measure of the potential solute hold-up can be estimated heuristically. Assuming relative destabilization (attachment) on contact between the virus particles and the membrane surface and negligible reentrainment, it seems reasonable to propose a hold-up index $\bar{H}$ as follows:

$$\bar{H} \equiv 8\pi \times v_w p_f L / |\dot{V}_w| \quad (22)$$

Substituting for $|\dot{V}_w| = |-\langle v_z \rangle 4/R|$ for laminar flow in tubes and

$$p_f = t_T/t_1 = \frac{\Delta V_p / (2\pi R L v_w n)}{L / \langle v_z \rangle} \quad (23)$$

into Eq (22), the following single expression is obtained

$$\bar{H} \equiv |\Delta V_p / (Ln)| \quad (24)$$

VIRUS CONCENTRATION RESULTS

A summary of the results is presented in Table 1. Additional details on the materials and methods used in each experimental set are available in the literature (15-17). The symmetric semi-dense cellulose

acetate and asymmetric polysulphone membranes with inside diameters of less than 1.0 mm are arbitrarily called hollow fibers, while the asymmetric polyamide membranes which have an inside diameter greater than 1.0 mm are arbitrarily called capillaries.

The results presented in Table 1 for tap water virus concentration with the asymmetric polyamide capillaries represent a significant improvement over the cellulose acetate and polysulphone hollow fibers. Higher total average virus recoveries $\bar{R}$, higher average permeation coefficients $\bar{L}_p$, larger initial volumes V_f and lower initial virus concentrations C_{f0} , were obtained for the latter capillary study. The values for the average virus rejection R for the capillary membranes were much lower than previously measured, while average virus recovery $\bar{R}$ from backwashing was consequently much higher. Because of the low initial virus concentration for the capillary polyamide membranes, it was not expected that the permeability L_p (or flux v_w , cm sec⁻¹) would be reduced during dehydration. (Assuming the diameter of a poliovirus to be about 300 Å, the total projected area (monolayer) for the highest initial virus input, $A_p = (71 \times 10^5) (\pi/4) (300 \times 10^{-8})^2 = 5 \times 10^{-5}$ cm² is negligibly small relative to the 5×10^3 cm² area available for permeation. Thus fouling due to the virus itself is not expected.) See later discussion where the reduction in on hydrodynamics $v_w (= L_p \Delta P / \ln mm / 3600)$ is attributed to suspended solids in the Jerusalem tap water rather than the seeded virus. Buffered backwash solutions containing glycine or beef extract were superior in recovering missing (i.e. adsorbed) virus after concentration by ultrafiltration than previously used deionized water.

HYDRODYNAMIC ANALYSIS

A crucial element in optimizing mass transfer processes is the ability to understand and control the fluid dynamics of the system (33). In comparing membrane performance for concentrating viruses with different modules, it is important to compare their hydrodynamic conditions. This may help in choosing the best membrane (i.e. high intrinsic flux and low or high virus adsorptivity) with optimum geometry (i.e. high fiber diameter and length) and optimum fluid dynamic conditions (i.e. with high or low wall shear rate and low pressure loss along the flow path).

Laminar Flow in Circular Tube

Based on the manufacturer's supplied information and on the chosen operating conditions, the specifications for hollow fiber and capillary membrane modules are compared in Table 2. Using these specifications several hydrodynamic parameters are calculated and presented in Table 3 for laminar tubular flow (34).

All the Reynold's numbers N_{Re} in Table 2 are clearly below 2100 and are well within the laminar regime, although N_{Re} for the capillary unit is more than 4 times that of the other units. This higher value of N_{Re} is misleading as it does not explain the higher $\bar{L}_p$ for the capillary unit presented in Table 1. Because of the large capillary inside diameter, a low value of the shear rate at the membrane-solution interface γ_w results (see Table 3). The higher $\bar{L}_p$ can rather be explained by the higher molecular weight cut-off and relatively high intrinsic water permeability for polyamide membranes as compared to most other (not cellulose acetate) membranes (35,36). Also, since the polyamide capillaries have a larger inside diameter (~1.5 mm) than the polysulphone hollow fiber (~0.10 mm), a lower internal pressure loss ΔP_f and

conceivably a higher average transmembrane driving force $\Delta P_{\text{in mm}}$ should be operative (since for laminar flow the pressure-loss is inversely proportional to the tube diameter to the fourth power at constant volumetric flow rate (4)). From Table 3, it can be seen that the value of ΔP_f is the smallest for the capillaries but due to its high entrance pressure $P_f = 2.7$ atm, the HIDPIO module has the highest $\Delta P_{\text{in mean}} = 1.526$ atm.^o Since pressure losses along the flow path ΔP_f is less for laminar than for turbulent flow, it is advisable from this point of view to operate under laminar conditions (34,37).

Solute Removal During Concentration

Virus Removal

In Table 1 it can be seen for the polyamide capillaries that most of the virus is held-up in the system and finally recovered by backwashing. Thus $\bar{B}$ values are close to $\bar{R}$, and $\bar{R}$ values are expectedly low. Since no virus was detected in the permeate, it is assumed that the well-known concentration polarization phenomenon due to permeation is responsible for this high virus loss. The low shear rate, long flow-path, and high average number of fluid passes P_f , for the BMR 500515 unit may explain the high tendency to adsorb or hold-up virus particles (38). It should however be noted that the average number of virus passes P_s is probably not equal to the average number of fluid passes P_f due to the polarization and retardation (R_s) of the viruses adjacent to the membrane-solution interface.

In Table 4, the hold-up index $\bar{H}$ is compared with $(1-R)$ as a measure of virus hold-up during concentration. Wide possible variations due to experimental errors in both $\bar{B}$ and $\bar{R}$ could explain the relatively poor linear correlations of $\bar{B}/\bar{R}$ with $\bar{H}$ and $(1-R)$, i.e. $r = 0.673$ and 0.734 respectively. The linear correlation between $(1-R)$ and $\bar{H}$, however, is good ($\bar{r} = 0.921$). This suggests the use of $\bar{H}$ as a good qualitative estimate of virus hold-up during concentration.

We have previously shown that during concentration poliovirus 1 does not noticeably aggregate as a result of orthokinetic flocculation, but is removed from the feed water onto the membrane at about the same rate as the water permeation flux (17). Eq (5) was used to show that $\bar{R} \rightarrow 0$ and $\bar{C} \rightarrow 1$ or constant virus concentration during dehydration.

Suspended and Dissolved Solids Removal

Virus adsorption onto and protection by suspended and dissolved solids is well known (11). The presence of these solids can also clog the membrane and reduce the permeation flux v_w during the concentration process (17).

Two control experiments with and without $0.3 \mu\text{m}$ prefiltration of the tap water feed (the tap water feed was filtered with a $0.3 \mu\text{m}$ Grade B tubular filter (Balston, Inc., Maidstone, England, U.K.) prior to seeding with the poliovirus 1) [Exps. 41 and 42] were conducted to determine (1) the variation of suspended solids (measured by reduced turbidity, $\bar{T}$) [turbidity was measured with Model 2100 A Turbidity meter Hach, Colorado] and dissolved solids (measured by reduced dissolved organic carbon, DOC) [DOC was measured with a Total Organic Analyzer (Oceanography, and a UV attachment as per Dr. Sontheimer, Karlsruhe, Germany)] in the feed during dehydration, and (2) their combined effect on permeation flux, v_w .

The results are summarized in Figs. 4, 5, and 6. In both Figs. 4 and 5, a concentration parameter ($\bar{r}$ and DOC) is plotted against a reduced feed volume $\bar{V}_f$ according to Eq (6). The linear correlations $\bar{r}$ are acceptable except for the case with a prefilter in Fig. 4. According to Eq (6), the slopes equal R . Thus from Figs. 4 and 5, the following ratios are obtained:

$$[R(\text{no prefilter})/R]_{\bar{r}} = 3.81$$

$$[R(\text{no prefilter})/R]_{\text{DOC}} = 1.00$$

Thus, the 0.3 μm prefilter has no effect on the removal of DOC from the tap water feed during dehydration. Since $R_{\text{DOC}} \approx 0.17$, the DOC is removed slightly slower than the permeation flux v_w . On the other hand, prefiltering the feed does effect turbidity removal during dehydration. The high value of $R = 0.419$ for turbidity removal without prefiltration suggests that the suspended particles are only being removed from the feed at about half the permeation rate or that v_w is decreasing faster than v_s causing R to increase. See arguments for v_w below.

Solvent Removal During Concentration

Temperature Effects

Because the pump imparts energy into the feed fluid during recycle, the temperature increases from about 20°C to 36°C when concentrating 100 l. This is very fortunate, since the relative flux increases near the end of the experiment due to this temperature increase. This compensates the expected flux-decline due to membrane clogging especially near the end of the experiment and results in better performance than expected. Without temperature compensation, the inverse reduced flux $\bar{v}_w(T)^{-1}$ shows a maximum versus $\bar{V}_p$ in Fig. 7. With temperature compensation, $\bar{v}_w^{-1}$ shows a linear correlation fit with $\bar{V}_p$ of $\bar{r} = 0.9942$ as predicted by Eq (4).

Similar plots with good correlations for inverse flux $\bar{v}_w^{-1}$ versus $\bar{V}_p$ with and without a 0.3 μm prefilter are shown in Fig. 6. The following ratios are obtained for the cake

$$k_1(\text{no prefilter})/k_1 = \hat{\omega}_w(\text{no prefilter})/\hat{\omega}_w = 1.414$$

and for the membrane

$$k_2(\text{no prefilter})/k_2 = r'(\text{no prefilter})/r' = 0.838.$$

Thus, without a prefilter the relative resistance of the cake is high (i.e. a high flux-decline is expected). A slightly higher membrane resistance r' results for the case with a prefilter. Since from Figs. 4 and 5, the 0.3 μm prefilter removes turbidity and not DOC, the change in cake resistance is clearly turbidity related. This is due to the fact that turbidity continually increases with dehydration for the case without a prefilter (i.e. $R = 0.419$).

Virus Retardation and Dispersion Analysis

The response to an impulse of injected poliovirus (1 ml of 3.33×10^8 (PFU) into the flowing fluid ($Q_{in} = 5.94$ ml/sec) prior to the BMR 500515 module is shown in Fig. 8. The value of the dispersion coefficient D is

obtained from the dispersion number $\alpha = D/(\langle v_z \rangle L) = 0.0117$. $\langle v_z \rangle$ is usually chosen as the average velocity along the flow path L for the fluid carrying a conservative tracer. From an estimate of the area under the curve in Fig. 8 only about 1.4% of the original virus eluted. Assuming permeation forces are responsible for this 98.6% virus loss from the flowing stream, an average permeation flux $v_w = 5.94/(2\pi RLn) = 5.94/(2\pi \times 0.075 \times 50 \times 210) = 1.200 \times 10^{-3} \text{ cm s}^{-1}$. Thus from Eq (10), $R_1(0) = 0.075 - (1.2 \times 10^{-3})(50)/(1.601) = 0.0375 \text{ cm}$. $R_1(50) = 0.075$ and $b = 1.387 \times 10^{-2} \text{ cm}^{-1}$ in Eq (11). Using the concept of an average escape radius defined by Eq (12) $\langle R_1 L \rangle = 0.0541 \text{ cm}$ which when substituted into Eq (15) gives an average escape velocity $\langle v_z \rangle_{es} = 0.74 v_z^0$. This velocity is the average escape velocity of the fluid and particles escaping out of the porous tubes (see Fig. 1). For $v_z^0 = 3.202 \text{ cm s}^{-1}$, $D = (0.0117)(0.74 \times 3.202)(50) = 1.386 \text{ cm}^2 \text{ s}^{-1}$. From Eq (17) $D = (\langle R_1 L \rangle v_z^0)^2 / (192D) = (0.0541 \times 3.202)^2 / (192 \times 1.386) = 1.128 \times 10^{-4} \text{ cm}^2 \text{ s}^{-1}$. This "escape" value of D for poliovirus 1 in a permeation tube is very high relative to that predicted by the Stokes-Einstein relation (17) of $1.43 \times 10^{-7} \text{ cm}^2 \text{ s}^{-1}$ or that quoted in the literature (39,40) of $1.5 \times 10^{-7} \text{ cm}^2 \text{ s}^{-1}$. A previous estimate of D using the Gel-Polarization model gave (17) $1.59 \times 10^{-5} \text{ cm}^2 \text{ s}^{-1}$. These large deviations have been explained via the tubular pinch effect (41). Although the physical cause of the tubular pinch effect is still somewhat controversial, the result of it is that particles migrate away from the tube axis at a much higher rate than would be expected from fluid viscosity arguments. Additional details can be found in Reference (41).

The retardation, for $P\bar{b} = R \bar{v}_w / D = (0.075)(1.2 \times 10^{-3}) / (1.128 \times 10^{-4}) = 0.798$ is negligible according to Eq (8). Experimentally this is confirmed from Fig. 8, where the retardation

$$R_s = \frac{t_{\text{solvent}}}{t_{\text{solute}}} = \frac{\theta_{\text{solvent}}}{\theta_{\text{solute}}} = \frac{0.938}{1.000} = 0.938$$

for the escape region. Using the value of the diffusivity for poliovirus 1 quoted in the literature of $1.5 \times 10^{-7} \text{ cm}^2 \text{ s}^{-1}$, $P\bar{b} = 600$ which when substituted into Eq (9) gives a retardation $R_s = 0.0067$. This suggests that the poliovirus 1 hardly moves axially and could be considered stationary during the time constant of the above experiment.

In conclusion, with reference to Fig. 1, particles that enter the high flux ultrafiltration flow tubes within the capture region ($R_1(0) < r < R$) are held in the module for later collection by backflushing. In BMR 500515 module, this constitutes greater than 98% removal of the virus. The remaining virus entering in the escape region ($r < R_1(0)$) is hardly retarded relative to the carrier fluid and appears to have an abnormally high apparent diffusivity. The RTD is Gaussian as expected (30).

From the above analysis with the BMR 500515 module, about 98.6% of the virus is removed from the feed fluid during one fluid pass. The final dehydration rates based on the data in Tables 1-3 are calculated in Table 5. For the tap water feed, the BMR 500515 module takes about 90 min to treat 100 ℓ in a 0.5 m^2 module.

The total fraction $f_T = \Delta N/N$ removed from the feed solution after P_f integer passes can be calculated from

$$f_T = [1 - (1 - f_1)^{P_f}] \quad (25)$$

where $f_1 = 0.986$ is the assumed constant removal fraction for each pass.

To get $f_T = 0.9999(9)$ only 3 fluid passes are needed. For similar total removals Eq (25) shows that if $f_1 = 0.9, 0.8, 0.7, 0.5, 0.3,$ and 0.1 then $P_f = 5, 7, 9, 15, 28,$ and $94,$ respectively.

From the above dispersion experiment and subsequent analysis, it appears that only 3 fluid passes of each initial volume are necessary for $f_T = 0.9999(9)$. The actual number $P_f = 1600$ is shown in Table 3. This amazing difference is due to the fact that the module is surface area limited. If for example, the largest commercially available BMR 500515 module is used (i.e. 5 m^2), then $\theta_T \approx 9$ mins with the actual P_f significantly lower to treat 100ℓ . 1900ℓ could conceivably be treated in about 2.85 hrs.

CONCLUSIONS

The differences in performance of 3 hollow fiber-capillary membrane modules in concentrating poliovirus 1 from deionized and tap water were compared and evaluated from a hydrodynamic viewpoint. Implicit in this study was that, by understanding and controlling the fluid mechanics of the modular system, optimum performance could be attained. The major conclusions are:

1. For the asymmetric membrane modules, permeation forces dominate within the hollow fibers causing solutes such as poliovirus 1, turbidity and dissolved organic carbon to move toward the membrane and out of the axial velocity field. After dehydration, the viruses are then recovered by backwashing.
2. The expected flux-decline due to membrane clogging is not observed due to a steady increase in fluid temperature from recycle pumping. This results in a relatively steady permeation flux through the concentration process.
3. The modified temperature compensated filtration equation was used to analyze permeation flux data with good linear correlations ($\bar{r} > 0.985$).
4. The retention efficiency parameter R was used to compare the relative removal rates of solute with solvent. In most cases R was close to zero implying a constant solute concentration during dehydration.
5. Comparison of a filtered tap water (0.3 microns) with an unfiltered tap water prior to seeding with poliovirus 1, gave:
 - a. the flux-decline rate with permeation was about 1.4 times higher without than with prefiltration,
 - b. the removal of turbidity during dehydration from the feed solution was about 3.8 times less without than with prefiltration,
 - c. prefiltration had no noticeable effect on the rate of dissolved organic carbon removal during dehydration.
6. A simple easy to measure empirical parameter called the hold-up index is proposed for estimating virus (or solute) hold-up during dehydration. It correlates well ($\bar{r} = 0.921$) with the actual virus hold-up $(1-R)$.
7. The response to an impulse of poliovirus through the BMR 500515 module was analyzed and gave the following results:

a. About 98.6% of the original poliovirus 1 number was held up in the module and did not elute during the experiment. Assuming this fractional removal/pass to be constant, all that is needed is about 3 fluid passes to obtain a maximum fractional removal of 0.9999(9). Thus the 1600 fluid passes (see Table 3) are needed only because the module is permeable surface area limited.

b. Using Taylor's dispersion analysis for the escape region only, an abnormally high apparent diffusivity for poliovirus is calculated.

c. Using convective polarization theory and a diffusivity for poliovirus from the literature, a retardation factor of 0.006(7) is estimated for the capture region. This confirms the experimental findings of large fractional virus removals during dehydration.

8. Virus concentration performance for the BMR 500515 module was found to be the best so far. Higher final average virus recoveries with higher average fluxes were obtained with this module than with the other two. It should however be pointed out that the HIDPIO module runs a close second and could with some modifications in operational procedures (such as using organic additives during backwashing) attain higher recoveries. Its lower flux may still be a limitation.

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NOMENCLATURE

a	= particle radius, cm
A _p	= total projected area of a monolayer, cm ²
b	= constant defined by Eq (11), cm ⁻¹
b'	= intercept from linear correlation fit
B	= average virus recovery from backwashing, %
C _b	= bulk particle concentration, moles cm ⁻³
C _f , C _{f0}	= mean particle concentration at z = z and z = 0, moles cm ⁻³
C	= C _f /C _{f0}
d	= hollow fiber inside-diameter, cm
D	= dispersion coefficient, cm ² s ⁻¹
DOC, DOC ₀	= dissolved organic carbon at z=z and z=0, mg/l
DOC	= DOC/DOC ₀
D, D _p	= binary diffusivity of solute derived from dispersion analysis and Stokes-Einstein relation respectively, cm ² s ⁻¹
D _w	= binary diffusivity of water at temperature T, cm ² s ⁻¹
E	= reduced C _f from RTD analysis
f ₁	= virus removal fraction for one fluid pass
f _T	= virus removal fraction for the whole experiment (P _f passes)
H	= empirical hold-up index defined by Eq (22), cm ²
k ₁ , k ₂	= constants defined after Eq (4).
k _B	= coefficient for Brownian motion, defined by Eqs (18) & (19).
L	= length of the porous hollow fiber, cm
L _p	= average permeability coefficient, cm hr ⁻¹ atm ⁻¹
m	= slope from linear correlation fit.
n	= number of hollow fibers
N _f , N _{fi}	= virus number at z=z and z=0
N	= N _f /N _{fi}
N _{Re}	= d<v _z >/ζ, Reynold's number
P _f	= number of fluid passes
P _f	= entrance pressure of the fluid, atm
ΔP _f	= internal pressure loss in the tube, atm
ΔP _{lmm}	= log mean driving force across the tube wall, atm
ΔP _c , ΔP _m	= pressure drop across the cake and membrane, respectively, atm
Pe	= R v _w /D, Péclet number
Q, Q _{in}	= overall volumetric flow rate at z=z and z=0, cm ³ s ⁻¹
Q _p	= volumetric permeation rate per pass, cm ³ s ⁻¹
r	= radial coordinate from axis of tube, cm
r'	= intrinsic membrane resistance, cm ⁻¹
r̄	= correlation coefficient
R	= inside radius of the tube, cm
R̄	= average total recovery, %
R _s	= retardation factor defined by Eq (8)
R ₁ (m)	= limiting trajectory radius at z=z
K	= fraction of the solute retained in the fluid
S	= active permeable surface area, cm ²
t ₁	= time for one fluid pass, sec
t _T	= time for whole experiment, sec
T	= temperature, °C
u _s	= axial velocity of solute, cm s ⁻¹
v _B	= Brownian velocity defined by Eq (18), cm s ⁻¹
v _s	= average permeation velocity of solute at z, cm s ⁻¹
v _w , v _{wi}	= average permeation velocity of solution at z=z and z=0, cm s ⁻¹
v̄ _w (T)	= v _w /v _{wi}
v̄ _w	= v _w R/D=pδ
<v _z >	= average fluid velocity in tube, cm s ⁻¹
<v _z > _{es}	= average fluid velocity in escape region, cm s ⁻¹

v_z^o	= maximum or centerline velocity at $z=z$, cm s^{-1}
V_f, V_{fi}	= reservoir volume of the feed during the experiment and initially, cm^3
$\bar{V}_f$	= V_f/V_{fi}
V_p	= accumulated volume permeated during the experiment, cm^3
$\bar{V}_p$	= V_p/V_{fi}
$\Delta \bar{V}_p$	= total final accumulated permeation volume, cm^3
w	= weight of dry solids/volume of filtrate, gm cm^{-3}
w_o	= constant w , gm cm^{-3}
W	= total accumulated weight of suspended solids deposited on the membrane, gm.
y	= radial coordinate from the tube wall, cm
z	= axial distance along the tube in the direction of solvent flow, cm

GREEK LETTERS

α	= $D/(\langle v_z \rangle L)$, reactor dispersion number
$\bar{\alpha}'$	= average specific cake resistance, cm gm^{-1}
γ_w	= sheer rate at the tube wall, s^{-1}
ζ	= kinematic viscosity of filtrate, $\text{cm}^2 \text{s}^{-1}$
θ	= reduced time from RTD analysis
θ'	= dummy angular variable in Eq (13)
μ	= dynamic viscosity of the filtrate, $\text{gm an}^{-1} \text{sec}^{-1}$
$\Delta \pi$	= osmotic pressure difference, atm
σ^2	= variance of the RTD
τ, τ_o	= feed turbidity during experiment and initially, F.T.U.
τ'	= τ/τ_o

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Table 1. Summary of Poliovirus 1 Concentration by Symmetric Semi-Dense Cellulose Acetate, Asymmetric Polysulphone and Asymmetric Polyamide Hollow Fiber-Capillary Membranes with Core Feeds

Membrane type	Initial feed volume V_f liter	Feed-water type	Average total recovery R %	Average rejection R	Average permeability coefficient L_p ml cm ⁻² h ⁻¹ atm ⁻¹	Average backwash recovery $\frac{B}{B}$ %	Backwash solution	No. of exps.
Symmetric ^b semi-dense cellulose acetate	5	deionized	84±9	0.759	0.599	36±17	deionized	11
Asymmetric ^c polysulphone	50	deionized	77±33	0.959	11.31	26±14	deionized	12
	5	deionized	61±22	0.963	13.25	46±22	deionized	4
	50	tap	52±14	0.846	7.61	38±19	deionized	4
Asymmetric ^d polyamide	100	prefiltered tap	106±42	~0.330	16.76	101±42	0.05M glycine at pH ±10.5	5
	100	tap	67±35	~0.270	13.61	65±34	0.05M glycine at pH = 10.5	5
	100	tap	67±19	-	13.34	48±24	1% beef extract at pH = 9.0	4
	100	tap	80±20	-	13.59	-	1% beef extract at pH = 9.0 with organic flocculation	5

^aThe reported errors are for 1 standard deviation.

^bResults from Ref. (15) with initial virus concentrations of $0.047 < C_f < 3000$ PFU ml⁻¹.

^cResults from Ref. (16) with initial virus concentrations of $0.0066 < C_f < 934$ PFU ml⁻¹.

^dResults from Ref. (17) with initial virus concentrations of $39 \times 10^{-5} < C_f < 71$ PFU ml⁻¹.

Table 2. Hollow Fiber Module Specifications

Module Memb. Type Overall Dimen. cm x cm	Manufact.	Fibers ^a			Circ. Rate ^b		Mol. Wt. Cut- off
		Numb. n	Dimen. i.d. x wall $\mu\text{m} \times \mu\text{m}$	Trans. Area $\text{cm}^2 \times 10^{-3}$	Q ml min^{-1}	N_{Re}	
Mini-plant 80 cellulose acetate 5.1 x 21.6	Bio-Rad Labs Richmond, CA, USA	6366	200 x 25	10	240-256	39-60	30,000
HIDPIO polysulphone 2.7 x 22	Amicon Corp. Lexington, MA, USA	1000	200 x 200	1	150-400	27-423	10,000
BMR 500515 Polyamide 7.5 x 52	Forschungs-Institut Berghof, GmbH Tubingen Germany	210	1500 x 200	5	3500	236	50,000

^aTransfer area supplied by the manufacturer. Dimensions measured by SEM (Cambridge Model S4-10, UK). Fiber number estimated from dimensions and transfer area.

^bReynold's number, $N_{\text{Re}} = d \langle v_z \rangle / \zeta$ where $d = \text{i.d.}$, fiber inside diameter cm, $\langle v_z \rangle = \text{average tube velocity, cm sec}^{-1}$, $\zeta = \text{kinematic viscosity, cm}^2 \text{sec}^{-1}$

Table 3. Calculated Hydrodynamic Parameters for Laminar Tubular Flow^a
Pressure Considerations^b

Module	Feed entr. P_{fo} atm	Inter. loss ΔP_{f} atm	Log mean Diff. ΔP_{Ln} mm atm	Shear $\dot{\gamma}_w$ Rate ^c at Membrane sec^{-1}	Number of ^d Fluid Passes P_{f}
Mini-plant 80	2.0	0.07185	0.964	1820	656
HIDPIO	2.7	0.3357	1.526	8488	14900
BMR 500515	2.0	0.011	0.995	838	1600

^aSee Reference (34) for analysis and assumptions of laminar flow through a circular tube. Input parameters such as n, Q, L and d are obtained from Table 2.

^bInternal pressure loss is given by $\Delta P_{\text{f}} = (128 L Q) / (\pi n d^4)$ where maximum Q was used. The log mean differential pressure ΔP_{Ln} mm = $(\Delta P_{\text{O}} - \Delta P_{\text{L}}) / \ln(\Delta P_{\text{O}} / \Delta P_{\text{L}})$ where $\Delta P_{\text{O}} = P_{\text{fo}} - 1$ and $\Delta P_{\text{L}} = P_{\text{fo}} - \Delta P_{\text{f}} - 1$.

^cFor a Newtonian fluid, the laminar flow shear rate at the membrane-solution interface $\dot{\gamma}_w = -32Q / (\pi n d^3)$.

^d $P_{\text{f}} = \bar{t}_T / t_1$ where $t_1 = \text{single pass time}$, and $\bar{t}_T = \text{average time for the experiment}$.

Table 4. Virus Hold-Up During Concentration

Module	Feedwater type	Fraction Recovered by Backwashing, $\bar{B}/\bar{R}$	Hold-Up Index, $\bar{H}$ cm ²	1 - R
Miniplant 80	deionized	0.429	0.04	0.24
HIDPIO	deionized	0.338	2.50	0.04
	deionized	0.754	0.25	0.03
	tap	0.731	2.50	0.15
BMR 500515	prefiltered tap	0.953	9.52	0.67
	tap	0.970	9.52	0.73
	tap	0.716	9.52	-

^aData are from Tables 1 and 2 and $\bar{H} = |\Delta V_p / (\bar{L}n)|$. A linear correlation of $\bar{B}/\bar{R}$ versus $\bar{H}$ and 1-R, and $\bar{H}$ versus 1-R gives the following ($y=mx+b'$):

	x	y	intercept	slope	corr.coef.
$\bar{H}$	$\bar{B}/\bar{R}$	$\bar{B}/\bar{R}$	0.525	0.036	.673
1-R	$\bar{B}/\bar{R}$	$\bar{B}/\bar{R}$	0.504	0.617	.734
1-R	$\bar{H}$	$\bar{H}$	0.065	12.870	.921

Table 5. Dehydration Rates for Ultrafiltration

Module ^a ($2\pi RLn$)10 ⁻³ (cm ²)	ΔV_p ^b ℓ	$v_w \times 10^3$ ^c cm s ⁻¹	Q_p ^d ml s ⁻¹	θ_T ^e min
Miniplant 80 (8.000)	5	0.160	1.28	65
HIDPIO (1.257)	50	4.795	6.03	138
	5	5.617	7.06	12
	50	3.226	4.06	206
BMR 500515 (4.948)	100	4.632	22.92	73
	100	3.762	18.61	90
	100	3.687	18.24	91
	100	3.756	18.59	90

^aActive surface area = $2\pi RLn$, cm² (see Table 2 for data).

^bAssume $\Delta V_p \approx V_{fi}$

^cObtained from Tables 1 and 3

^d Q_p = volumetric permeation rate per pass = $v_w(2\pi RLn)$, cm³ s⁻¹

^e $\theta_T = \Delta V_p \times 10^3 / (Q_p / 60)$, min.

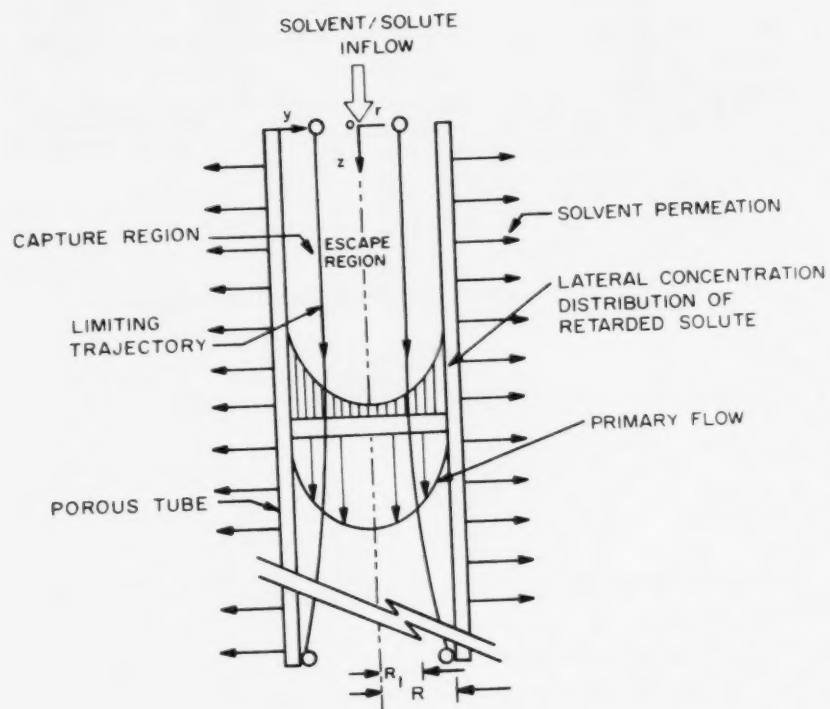


Fig. 1 Convective-Induced Polarization and Particle Capture in a Porous Tube

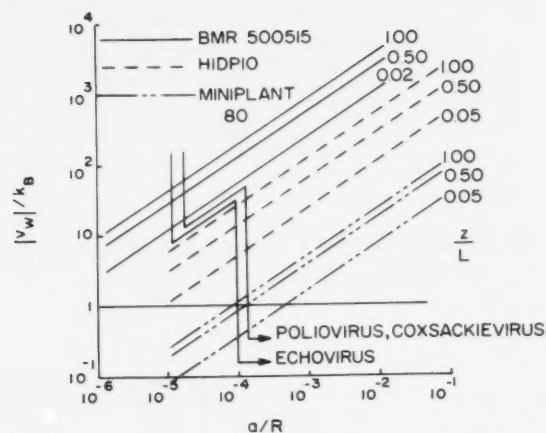


Fig. 2 Theoretical Comparison of Permeation with Brownian Motion for Three Hollow Fiber Modules

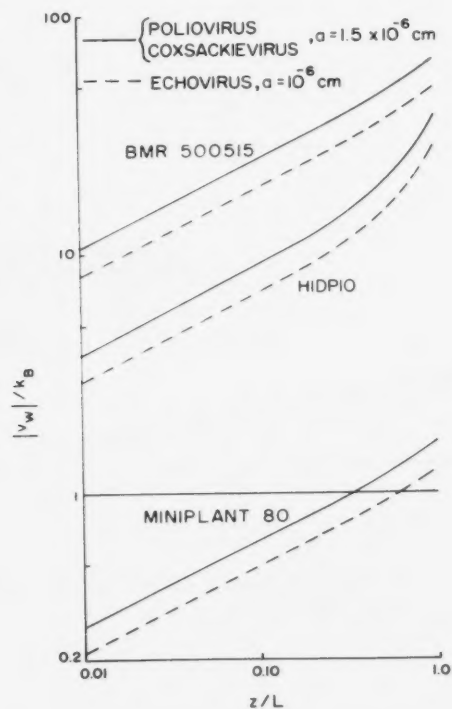


Fig. 3 Theoretical Comparison of Permeation with Brownian Motion for Three Enteroviruses and Three Hollow Fiber Modules

Fig. 4 Variation of Turbidity during Dehydration with the Berghof Module

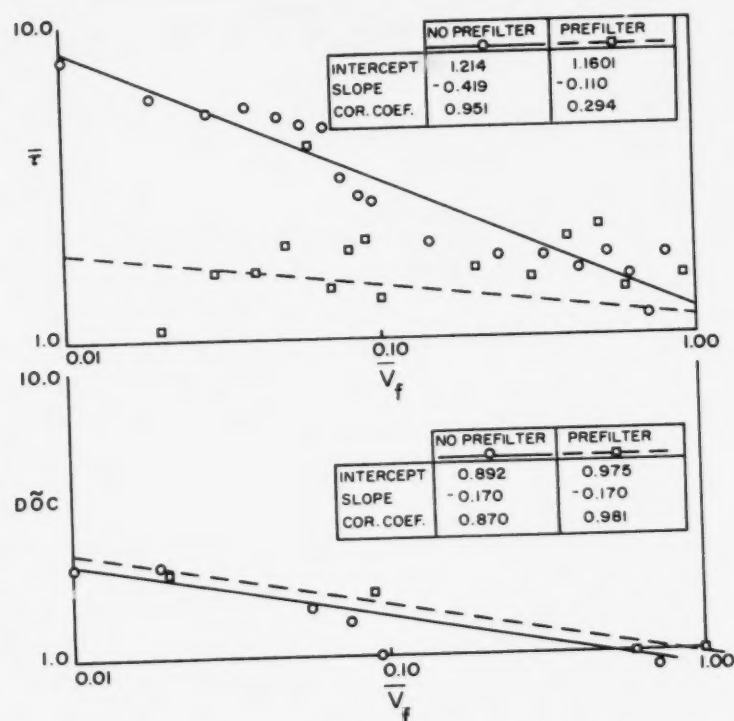


Fig. 5 Variation of Dissolved Organic Carbon during Dehydration with the Berghof Module

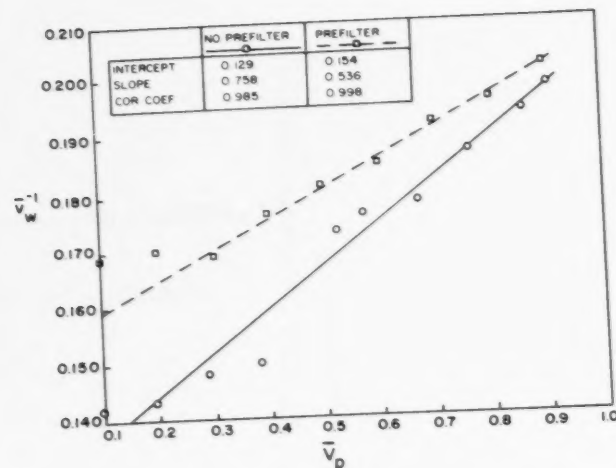


Fig. 6 Variation of Temperature Compensated Permeation Flux During Dehydration with the Berghof Module with and without a 0.3 micron Prefilter

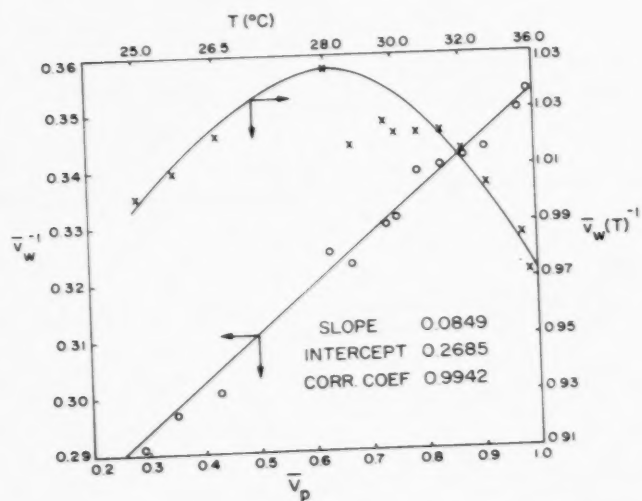


Fig. 7 Variation of Permeation Flux with and without Temperature Compensation During Dehydration with the Berghof Module

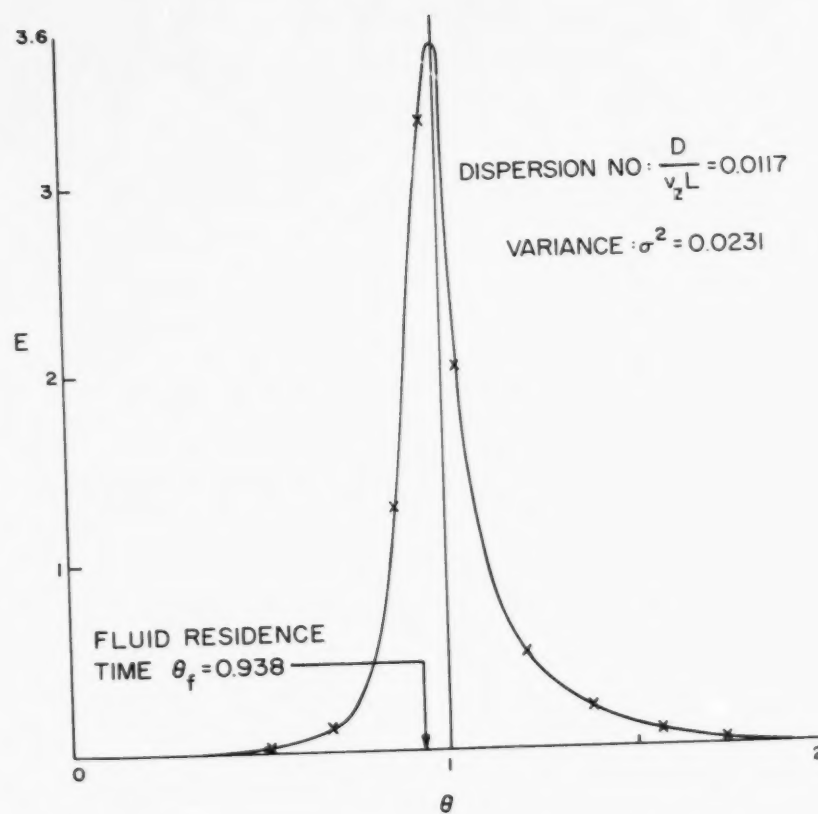


Fig. 8 Residence Time Distribution for a Poliovirus 1 Impulse for the Berghof Capillary Module at 25°C

WATHAN: AN URBAN WATER MANAGEMENT MODEL

By
Arun K. Deb,¹ Ph.D., P.E. and Richard L. Hillman²

INTRODUCTION

With the growing needs of urban water demand and the decreasing availability of good quality sources, it is apparent that efficient and economical management of water is an essential step in long-range water planning. Water pollution abatement requirements have resulted in the production of wastewater effluents of such quality that they may be considered sources of water for uses other than potable use. Where good-quality water is available in limited quantity and brought from long distance, it might be economical to conserve good-quality water for only potable use and to employ recycled water for other uses. Again, where good-quality water is not available, potable water would need to be produced from unprotected sources after extensive treatment to meet primary and secondary drinking water regulations.

Therefore, in areas where good-quality water is not available, or there is an overall shortage of water, developing a long-term economic water management plan will be complex. It might be necessary to consider more than one source of water (including wastewater reuse), and more than one distribution system, to supply water for potable and other uses. This study developed a comprehensive and easy-to-use systems model for analysis of long-term planning and management of a total urban water system.

In many cases, in order to conserve good quality sources, it may be useful and cost-effective to use more than one quality of water to meet various urban needs. In dual or multiple-supply systems, two or more grades of water would be supplied to consumers through separate distribution systems, according to the quality and quantity requirements for various uses. The balance between various grades of water will vary for each community depending on its residential, industrial, commercial, and public water quality and quantity requirements. In this model, a maximum of up to three qualities of water can be considered for analysis of a total urban water system.

For the purpose of this study, various grades of water in a multiple-supply system can be defined as follows:

- "Potable water" may be defined as water that is completely safe for long-term continuous human ingestion. This water should, as a minimum, satisfy primary and secondary drinking water regulations of the Safe Drinking Water Act.
- "Subpotable water" may be defined as water that is bacteriologically safe, may contain trace heavy metals and organics, but is not safe for long-term continuous human ingestion.

¹Principal Environmental Systems Engineer, Weston Designers-Consultants, West Chester, PA.

²Systems Analyst, Weston Designers-Consultants, West Chester, PA.

- "Nonpotable water" is not intended for, and may not be safe for, human ingestion.

WATER MANAGEMENT MODEL

In the future, in order to achieve an efficient and economical long-term water management plan, it would be required to consider more than one water source and more than one quality of water supply. This concept would produce a large number of alternative plans for supplying water in order to meet various demand needs of a city or a region. Some of these possibilities can readily be dismissed on the basis of engineering judgment and local conditions. However, a substantial number of alternatives will require full analysis and evaluation before selection of a final plan.

With a grant from the National Science Foundation, Weston has developed a WATER MANAGEMENT (WATMAN) model. The WATMAN model provides technical and economical evaluation of various alternatives of long-term planning and management of the total urban water system, and helps urban water and wastewater managers to find the cost-effective and implementable plan for their cities.

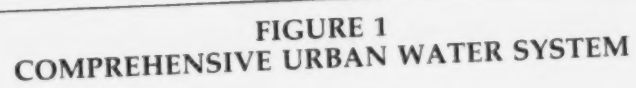
A general conceptual diagram of the WATMAN urban water system model is shown in Figure 1. A model of the quantity and quality aspects of an urban water network would incorporate the following three basic components:

1. Water sources, including recycled water.
2. Water demand by quality.
3. Water treatment (including treatment of wastewater after secondary treatment) and distribution (including more than one distribution system).

The model outline shown in Figure 1 may have numerous options, depending on: the quality and quantity of sources; potable, subpotable, and nonpotable water demand; and institutional and political considerations. This model provides the user with a tool to analyze various water supply system alternatives, including multiple distribution, depending on the composition of sources and demands. Water reuse would be an essential consideration in the future in developing any long-term water management plan for a city, industry, or a region. The WATMAN model will be particularly useful for analyzing water reuse management plans. Various components of the model are shown in Figure 2. At present, this model can only be used as a planning tool, rather than a design tool.

In analyzing a multiple or dual supply system, based on the sources and treatment requirements, the potable and subpotable/nonpotable system may be completely or partially separate. Total present worth of collection, transmission, treatment, and distribution of a water system of any configuration for conventional, dual, or multiple supply can be calculated using this model.

In formulating the mathematical model, parameters such as potable to total flow ratio, interest rate, annual capital and operation and maintenance (O&M) cost increase rates, and cost function are considered as variables.



1. Demand Development During Planning Period

Potable Demand
Subpotable Demand
Nonpotable Demand

2. Preliminary Design of Physical Components

Collection and Transmission
Treatment
Distribution
 Single
 Multiple
 Bottled

3. Cost of Components

Present Worth at Base Year
 Capital Costs
 O & M Costs
 Replacements Costs
 Salvage Value
Total Present Worth

4. Total Present Worth of the System

FIGURE 2
BASIC COMPONENTS OF SYSTEMS MODEL

The model has also been designed to accommodate regional management of water, with up to 10 cities in the region. In a regional water supply management analysis, one limited good-quality source may be considered to supply all potable demands of the region, and subpotable and non-potable demands can be drawn from local unprotected sources or from effluents of sewage treatment plants.

The system's primary function is to derive capital costs, operation and maintenance costs, salvage value, and present worth costs. The present worth costs are based on the user defined base or starting year for each unit process in the water supply management system. The system also provides summary cost information for the entire operation. Currently, the system contains a library of 36 unit processes that may be included in the design of a water supply management system.

SYSTEM OVERVIEW

Design Considerations

In designing the WATMAN system, many factors were taken into consideration. Key items include:

1. System Capability - Of major concern, care was taken to insure that the final product would have the capability and flexibility to effectively model a variety of water supply management system configurations.
2. Data Preparation/Structure - A simple and straightforward approach to creating job decks and running the system was essential from the user viewpoint.
3. Data Verification - Extensive data input verification was required to insure that the computer system would not be wasting time and resources by producing meaningless results or recognizing a system error resulting in abnormal termination.
4. Maintainability/Modularity - It was important to recognize the distribution logistics involved with the WATMAN system and provide an easy and effective approach to maintaining the system. The system is designed on a modular basis to support this thinking.
5. Computer System Resources/Compatibility - Every attempt was made to limit memory and peripheral device requirements such that a minimal computer system would be required. The language chosen was based on industry compatibility (ANSI FORTRAN IV).

System Capabilities

Based on the design considerations, the following capabilities exist:

1. The system is capable of handling a conventional water supply management system configuration as well as a multiple water supply management system configuration.
2. The system is capable of handling a regional water supply management system configuration serving up to 10 cities.

3. The design flow of a unit process may be derived from total average water demand or from interior residential water demand and population data.
4. Up to 50 unique unit processes may be included in the design of a water supply management system.
5. Multiple water supply management system designs may be processed in a submittal.
6. Presentation of various types of output may be suppressed.
7. Extensive input data verification is performed with easily understood error messages.

As with any system, constraints are placed upon it due to design considerations and the requirements of the computer language used. The most significant constraints are given in Figure 3. Computer hardware requirements are given in Figure 4.

MODEL EXECUTION

Two functional programs are used in the execution phase of the model. The Phase 1 program DEFCRT is used to define information about cities (such as name, population data, etc.), general information defaults, and unit processes with associated default values for variables which are placed in a "system default file". This data may be associated with user derived analysis or be data derived by other sources that the user feels will fit his situation. Phase 1 is referred to as the "system file creation" phase. This step is generally only performed once, but the user may redefine default variable values at any time.

For the Phase 2 program, WATHAN is used to derive costs associated with a unique water management system. The user defines his unique system by specifying information about cities (such as water demand), general information, and unit processes with associated data. User specifications and system default file data are read, water demand curves are derived, unit processes are invoked to calculate cost components, and information is output. Phase 2 is referred to as the "model execution" phase. This is a repetitive step for each unique water management system design the user wishes to consider.

INPUT DATA

The data required to execute the DEFCRT program can be grouped into four categories:

- General input.
- Community/city general input.
- Community/city demand type input.
- Unit process input.

The order of input is shown in Figure 5.

- 3 Water Supply Types
- 10 Communities/Cities
- 35 Years of Population Data Per City
- 50 Unit Processes
- 28 Variables Per Unit Process
- 10 Sets of Variables Per Unit Process
- 4 Unit Process Types
- 30 Year Planning Period
- Defined Order of Input Data
- Fixed Format Input (Generally 10 Columns)
- Batch Environment

FIGURE 3
SYSTEM DESIGN CONSTRAINTS

- 132 Column Line Printer
- Card Reader
- Disc File
 - Direct Access
 - 100 Records
 - Record Length: 200 Bytes
- Memory
 - Program DEFCRT: 50,000 Bytes
 - Program WATMAN: 65,000 Bytes

FIGURE 4
HARDWARE REQUIREMENTS

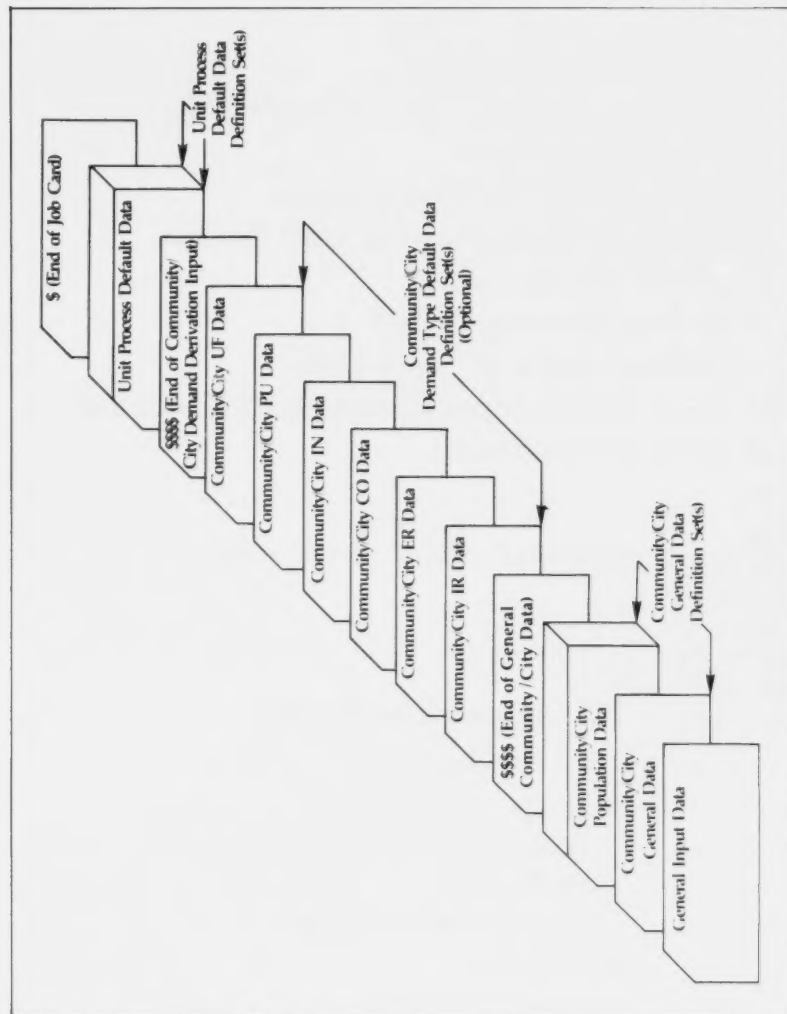


FIGURE 5
DEF CRT INPUT DATA

General Input

The general input is used to specify default values for the following:

- Planning period.
- Annual interest rate.
- Capital cost inflation rate.
- Operation and maintenance cost inflation rate.

In addition, the ENR index value and date that unit process cost default values are based on must be defined.

Community/City General Input

The community/city general input is used to specify general data for each city to be included in the user system environment. It consists of a set of cards for each city.

The general data card is used to specify the following:

- Unique city code.
- City name.
- Maximum to average daily flow ratio default value.
- Number of years of population data.
- Population data starting year.
- Number of people per residence default value.

Community/City Demand Type Input

This input is used to specify the potable, subpotable, and nonpotable default fractions and the interior residential default coefficient for a city for the following demand types:

- Interior residential.
- Exterior residential.
- Commercial.
- Industrial.
- Public.
- Unaccounted for.

An overview of WATMAN input data is given in Figure 6. Any input error will result in immediate program termination. All error messages are easily understood with no additional documentation required.

OUTPUT DATA

The output of the program consists of the following:

- General input data table.
- Demand curve tables for each city.
- Unit process input verification table.
- Unit process cost calculation tables.
- Cost summary table.

APPLICATIONS

Using this model, any water supply system new or existing can be analyzed for long-term planning and management. Depending on the sources and demand requirements, the user must identify the physical

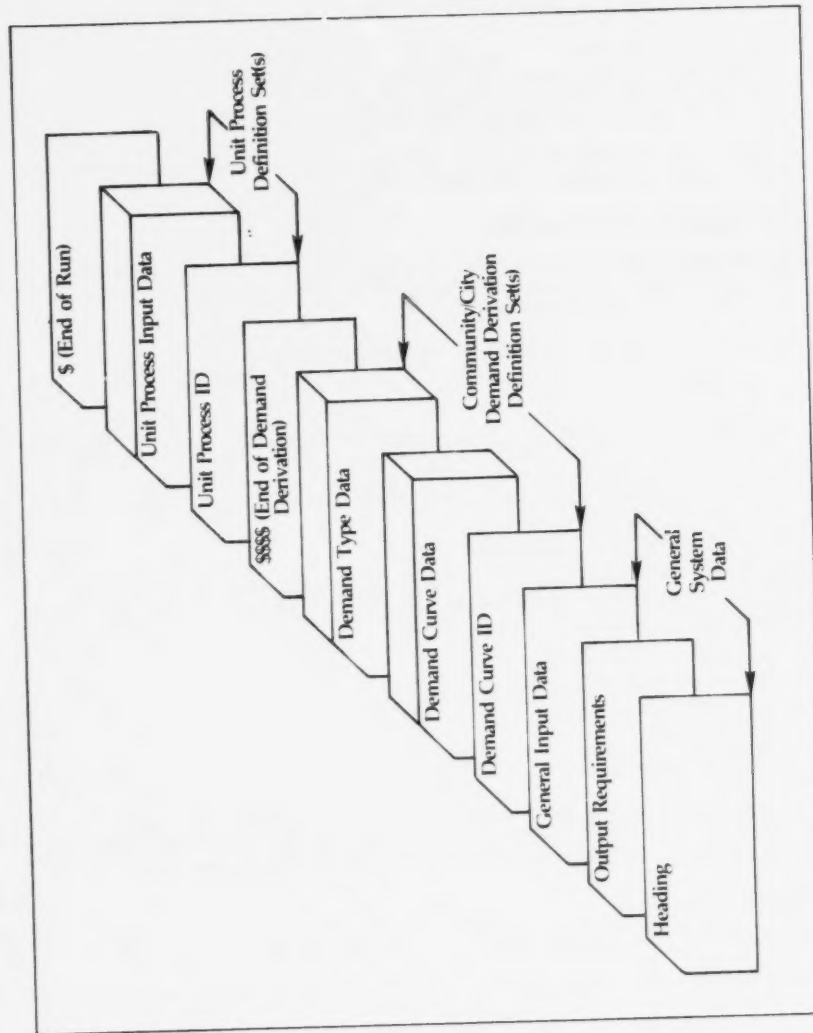


FIGURE 6
WATMAN INPUT DATA

configuration of various alternative water supply systems to be analyzed, proceeding from source to distribution. Several alternative systems should be analyzed in order to determine a cost-effective implementable solution.

Many hypothetical water systems under new and existing conditions have been studied using this computer model for conventional and dual water supply systems. Sensitivities of potable/total flow ratios in a dual system, inflation rates, and population densities on the costs of conventional and dual systems are studied. The computer output includes O&M and salvage values, and total present worth of each system. Common examples of reuse of wastewater for conservation of good quality sources are shown in Figures 7 and 8.

In an industrial city, where a large portion of the municipal water demand is for industrial needs, a dual supply can be effectively used with one nonpotable supply for industries and a potable supply for other uses. The industrial nonpotable supply can be derived from the same source with only existing treatment, or from reclaimed municipal wastewater with further treatment. Figure 9 shows a simple schematic of alternatives for an existing system on a polluted river which has been analyzed with this model.

In regional management of water, good quality-limited sources can be conserved and used for potable supply only through a regional water treatment and distribution system. The water for nonpotable uses can be obtained from reclaimed water (Figure 10).

CONCLUSIONS

The systems model developed in this study has been applied to several hypothetical and actual test case studies. The model is a powerful tool for technical and economical analysis of various feasible alternatives of water supply management for urban areas. This model can analyze any configuration of water supply systems, including reuse, using conventional, dual, or multiple supply systems. This method can be used in any specific case by putting the proper values of the various costs and other economic parameters in the systems model.

ACKNOWLEDGEMENT

This study was supported by a grant (ENV 76-18499) from the National Science Foundation.

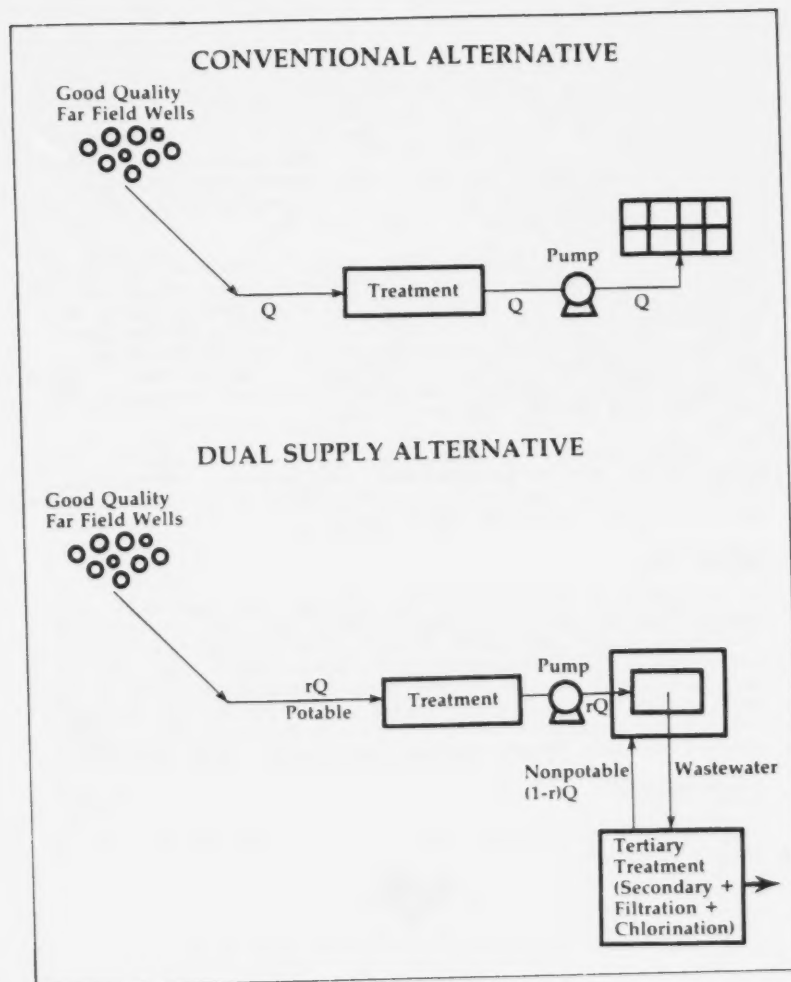


FIGURE 7
SYSTEM 1-2
CITY: 100,000 POPULATION

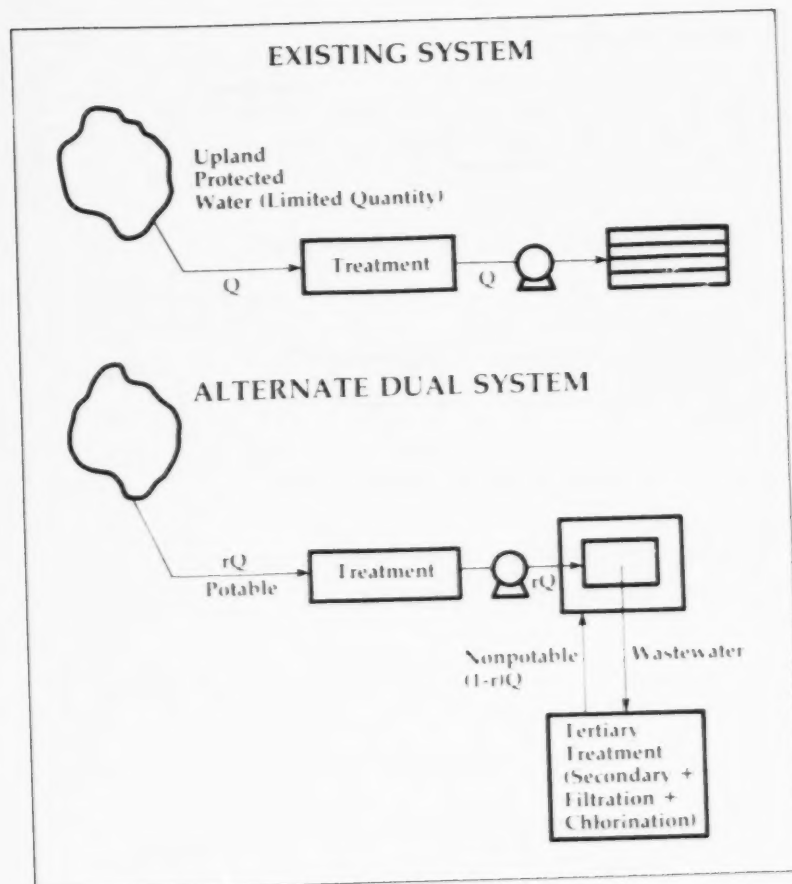


FIGURE 8

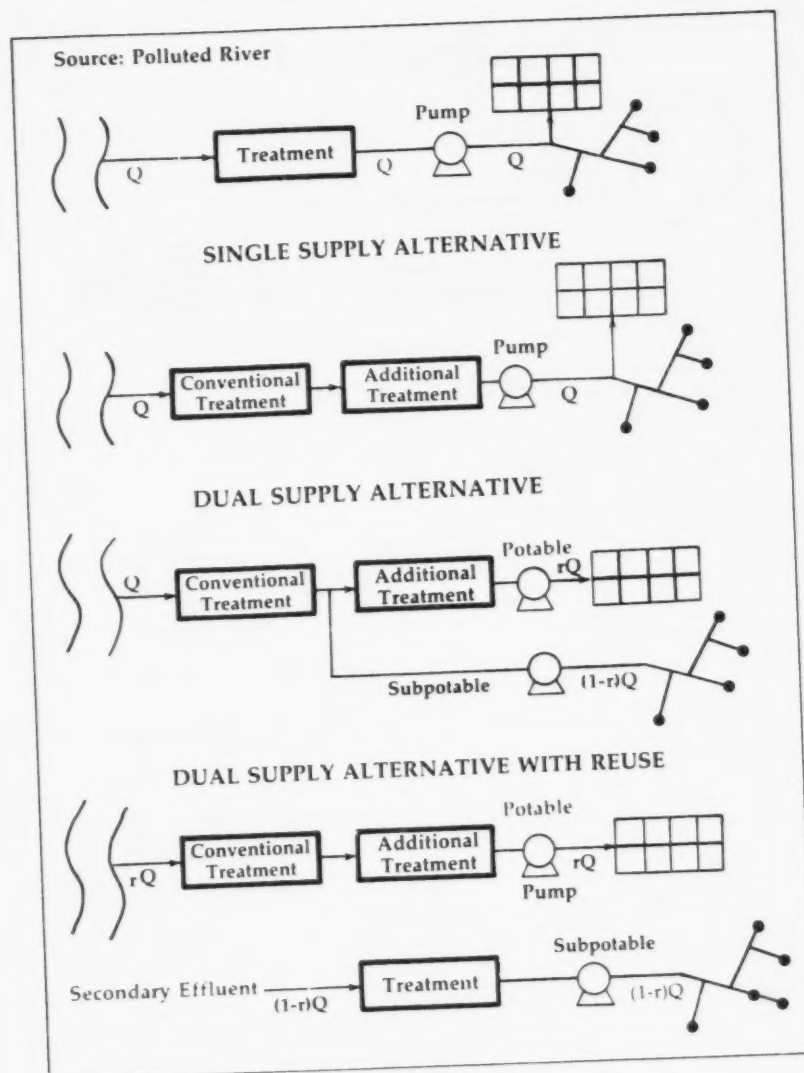


FIGURE 9
EXISTING SYSTEM
FOR INDUSTRIAL USES

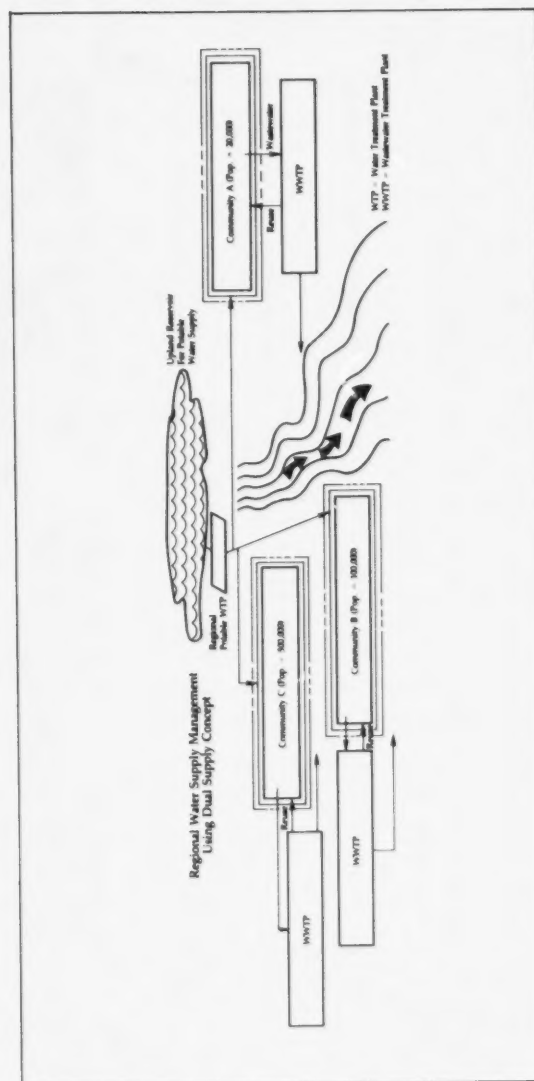


FIGURE 10
REGIONAL WATER SUPPLY
MANAGEMENT USING
DUAL SUPPLY CONCEPT

WATMAN: AN URBAN WATER MANAGEMENT MODEL **Arun K. Deb Richard L. Hillman**



OBJECTIVE

To develop a methodology for the evaluation of various alternatives in long term planning and management of total water systems for urban areas.

CONCEPT

Can effective uses of all available water sources including rivers, with up to three types of supply are considered.

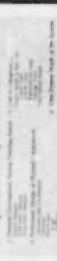
CONCLUSIONS

The model developed is a powerful tool for the analysis of total urban water management.

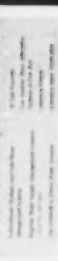
Total Urban Water System



Components of Systems Model



Current System Capabilities



*** Applications**



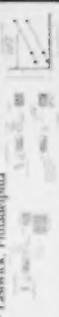
*** Industrial**



*** Regional**



*** Laswick, Philadelphia**



EVALUATION OF TWO WASTEWATER REUSE ALTERNATIVES BY SIMULATION

Larry D. Brown
Operations Research Analyst
Dallas Water Utilities

Sam A. Gray
Operations Research Analyst
Dallas Water Utilities

Dan A. Brock
Operations Research Analysis Manager
Dallas Water Utilities

INTRODUCTION

Indirect municipal reuse of wastewater effluent has long been of concern to downstream cities, for many receive dilute wastewater effluent from upstream cities. With growing population, the demand for purified water has increased. As a result, upstream cities have had to seek expanded water supplies. By virtue of improved techniques for purification and wastewater treatment, direct municipal reuse or recycling of water has become more competitive economically with other methods of expanding water supplies such as reservoir construction. When considering the feasibility of reuse locally, a city must consider several factors. These include the cost of upgrading wastewater treatment processes, and the cost of new purification techniques if they are required. Also, in most cases, there is added pumping cost. In addition, a city must determine the amount of effluent which it can safely reuse. Reuse of wastewater effluent, in most cases, is limited by the dilution capacity in the natural environment, in stream or lake. Finally, public acceptance of the reuse plan must be obtained. Paramount in the acceptance of any reuse plan is the use of natural processes in water purification, as treated effluent water is released into streams or lakes.

The purpose of this study was to examine two possible methods by which the City of Dallas could employ wastewater reuse, the examination centered on determining if the wastewater would cause excessive algae growth at the purification plant raw water source, whether lake or stream.

BACKGROUND

Dallas, Texas receives its water supply primarily from four lakes: Lake Grapevine, Lake Lewisville, Lake Ray Hubbard, and Lake Tawakoni. The Elm Fork Purification Plant (200 mgd) and the Bachman Purification Plant (125 mgd) take water from the Elm Fork of the Trinity River which has been released from Lakes Grapevine and Lewisville. The East Side Purification Plant (250 mgd) takes water directly from Lakes Ray Hubbard and/or Tawakoni. This arrangement is shown graphically in Figure 1. Dallas disposes of its wastewater effluent in the main stem of the Trinity River, well below any local raw water supply intakes.

One method that Dallas could employ to augment its water supply through reuse would be to divert water from the main stem of the Trinity River below the wastewater effluent discharges, and pump this water to the Elm Fork of the Trinity upstream from purification plants raw water

intakes. The required lake releases into the Elm Fork could then be reduced by the amount of the recycled water.

A second water reuse alternative would be to pump wastewater effluent into Lake Ray Hubbard. Garland, a suburb of Dallas, disposes of the effluent from its Rowlett Creek plant in the Rowlett Creek arm of Lake Ray Hubbard. This procedure contributes wastewater to the water supply of Lake Ray Hubbard. Garland proposes to abandon the Rowlett Creek plant and divert its sewage to the Duck Creek Wastewater Plant, where the effluent load is not dumped into Lake Ray Hubbard. To compensate for water lost as a result of this diversion, Dallas has proposed that the Duck Creek Plant pump some of its effluent back into Lake Ray Hubbard. Details of this proposal are shown in Figure 2.

The computer models used in this study were developed under two previous studies designed to determine the water quality of water bodies (rivers and lakes) in the Dallas area. The purpose of the river study (Metro-politan Water System Operation: Continuous Modeling of Flow Quantity and Quality in the Supporting River Basin, reference 1) was to develop a continuously varying flow and water quality model calibrated for the upper Trinity River. The Lake Ray Hubbard Eutrophication Study (reference 2) attempted development of a eutrophication model that would describe the relationship between nutrient loads and phytoplankton growth in water bodies located in the North Central Texas region near Dallas.

LAKE RAY HUBBARD STUDY

Eutrophication Study Objectives

The Lake Ray Hubbard Eutrophication Study focused on determining the amounts of aquatic plant nutrients (primarily nitrogen and phosphorus compounds) entering the lake, the fate of these nutrients within the lake, the factors limiting planktonic algal growth in the lake, and the response of the lake to the nutrients. In addition, information was obtained on the amounts of nutrients derived from various types of land use. Finally, some knowledge was obtained about the relationship between algal populations in Lake Ray Hubbard, and taste and odor problems in the drinking water from the Dallas Water Utilities East Side Water Purification Plant.

Model Development

The primary purpose of the modeling effort was to provide a basis for estimating the impact of alterations in the nutrient load to Lake Ray Hubbard upon eutrophication related water quality variables within the lake. After reviewing the various eutrophication models then (1977) available, it was decided that the best approach to take was that of using the basic structure of the LAKE I model, developed under a grant from the Environmental Protection Agency by Thomann, DiToro, O'Connor and Winfield at Manhattan College (reference 3). The LAKE I model attempts to formulate, using a series of differential equations and a mass balance approach, the major nutrient fluxes and the physical, chemical and biological processes that influence the concentration of algae (expressed as chlorophyll A) present in the water. It considers the different chemical species of nitrogen and phosphorus as potential limiting factors. Ambient temperature and available solar energy together with nutrients determine the phytoplankton growth rate. Herbivorous and carnivorous zooplankton are

modeled explicitly to complete a simple food chain. The model is vertically stratified into three horizontally mixed segments with appropriate parameters for sinking and upwelling of nutrients and biota.

When the LAKE I model was first received, it was not directly applicable to Lake Ray Hubbard. To satisfy the concept of horizontally mixed segments, only the main body of the lake was modeled. In the original LAKE I model, the sediment layer was modeled only as a sink for nutrients and detritus. However, in a shallow lake like Ray Hubbard, nutrient regeneration from the sediments has a significant effect on long-term nutrient levels. A procedure was written and incorporated into LAKE I which provides for nutrient regeneration and release to the overlying water via diffusion. To facilitate program modification and for ease of input and output, the LAKE I program was adapted to the IBM Continuous System Modeling Program.

Once modifications were complete, the model was calibrated to 15 months of data collected on Lake Ray Hubbard. Algae concentrations were measured as chlorophyll A concentration and as number of organisms. The algae counts were identified to the genus level. Over the sampling period, algae counts (average of lake main body samples taken on one day) varied from 4,000 to 22,700 cells/ml, while chlorophyll A ranged from 2.4 to 11.6 ug/l. Calibration consisted of adjusting model parameters until simulated concentrations of chlorophyll A, and nitrogen and phosphorus, agreed with measured values. Parameters adjusted include algae growth kinetic parameters, nutrient hydrolysis rates, zooplankton growth and feeding rates, and sediment nutrient interaction parameters. Although only an approximate calibration resulted, the model was in our view able to adequately simulate the major features of the observed chlorophyll A concentrations.

Several conclusions reached in the Lake Ray Hubbard Eutrophication Study are enumerated below:

- 1 Lake Ray Hubbard is receiving excessive amounts of aquatic plant nutrients from its watershed. These nutrients are stimulating the growth of algae which impair water quality, impairment primarily manifested by periodic objectionable tastes and odors in the domestic drinking water produced by the Dallas Water Utilities East Side Water Purification Plant.

- 2 Phosphorus is a primary chemical element controlling algae growth in the main body of Lake Ray Hubbard.

- 3 Nonpoint sources contribute 63.6 percent of the phosphorus (150,185 kg) and 41 percent of the nitrogen (237,981 kg) yearly load to Lake Ray Hubbard. The point sources (the sewage treatment plants of Plano, Garland, Rockwall, and Wylie) contribute 35.7 percent of the phosphorus (84,261 kg) and 37.7 percent of the nitrogen (202,820 kg) yearly loads. Atmospheric fallout contributes 0.7 percent of the phosphorus (1,569 kg) and 18 percent of the nitrogen (96,730 kg) load.

- 4 Highway causeways and bridges divide Lake Ray Hubbard into several arms and a main body, with restricted exchange of water between them. This results in each arm having a different water quality dependent on its nutrient input. Rowlett arm, which receives the treated wastewater discharges from Plano and Garland, shows highly degraded water quality. Muddy Creek arm has water quality which is somewhat

less degraded than that of Rowlett arm. Muddy Creek arm receives small amounts of treated municipal wastewater from the town of Wylie. The East Fork arm receives wastewater from the Rockwall sewage treatment plant, and has water quality similar to that of the Muddy Creek arm. The main body of the lake has the best overall water quality. The concentration of phosphorus in the wastewater which is discharged to tributaries which enter the arms of the lake is significantly reduced within each arm, thereby improving the overall water quality within the main body.

5 The relationship between algal numbers and types in raw lake water, and tastes and odors in the treated water derived from the lake, is not yet understood. No direct correlation could be found from this study which would be indicative of a relationship between nutrient loads and taste and odor characteristics of finished water from Lake Ray Hubbard. Although the relationship between algal types and taste and odor problems is not clear, evidence exists which suggests that blue-green algae of the genus *Oscillatoria* are at least partially responsible.

Evaluation of Reuse Feasibility

The modified LAKE I model was used to evaluate the management reuse proposal described below.

There were four model simulations of the main body of Lake Ray Hubbard made during this study. In all cases the treated effluent from the City of Garland Duck Creek Wastewater Treatment Plant was injected directly into the main body of the lake, as opposed to the current practice of injecting the treated effluent into a tributary of the lake (see Figure 2). Two levels of wastewater treatment were assumed for the Garland Duck Creek Plant effluent. The first assumes 1978 treatment levels (30 mg/l biochemical oxygen demand, 30 mg/l suspended solids, and 18.5 mg/l ammonia) with nutrient concentration loadings based on field measurements of treated effluent. The other level of treatment assumes the wastewater treatment plant has been upgraded to achieve a 10 mg/l biochemical oxygen demand and a 10 mg/l total suspended solids effluent. The other runs included the same levels of treatment but with different flow rates of injected effluent. A breakdown of nutrient loads into different chemical species for each run is given in Table 1. All other model inputs can be found in the Lake Ray Hubbard Eutrophication Study, Volume 3 (reference 3). All of the simulations were run for a period of ten years to investigate long term response of phytoplankton growth. The maximum phytoplankton growth simulated is also listed in Table 1.

The model predicted that at 1978 treatment levels the chlorophyll A concentration would reach 23.5 ug/l (micrograms per liter) for an effluent discharge of 5 mgd (million gallons per day) and 56.6 ug/l for a discharge of 20 mgd. For an effluent upgraded to 10 mg/l biochemical oxygen demand and 10 mg/l total suspended solids effluent, the model predicts a maximum chlorophyll A concentration of 20.6 ug/l for a discharge of 5 mgd, and a maximum of 42.8 ug/l for a discharge of 20 mgd. The maximum yearly chlorophyll A measured in the main body of Lake Ray Hubbard during the previous study was 11.6 ug/l. The increase in chlorophyll A predicted by the model indicates that the effluent should not be discharged into the main body of Lake Ray Hubbard, as this increase may cause unacceptable tastes and odors in the water drawn from the lake for municipal water supply. While chloro-

phyll A is not an excellent indicator of eutrophication because of its variable content in different algae species, it does correlate approximately with algal biomass. Work is now proceeding on a better lake model which will compute three dimensional transport resulting from wind induced circulation, and which will contain an ecosystem model with three classes of algae including blue-green algae.

Discussion:

If the main body of Lake Ray Hubbard is to be supplemented with treated wastewater without stimulating excessive algae growth, a better quality wastewater effluent is needed. A combination of additional lake data and improved simulation methods will be employed to determine how much nutrients would need to be reduced before reuse in the main body of Lake Ray Hubbard would be advisable.

ELM FORK TRINITY RIVER STUDY

River Model Development

The Dallas Water Utilities River Quantity/Quality Model was developed under a three year federal grant (OWRT) and was completed in March 1978. The model was developed to simulate major processes affecting water quality in river systems. The model simulates phytoplankton growth as a function of light, temperature, and available nitrogen and phosphorus. Other quality variables simulated are dissolved oxygen, biochemical oxygen demand, and suspended solids.

Quantity and quality portions of the model were calibrated for the Trinity River near Dallas. Flow rate parameters were adjusted based on measured flow rates and observed wave front propagation rates. Water quality parameters were calibrated during low flow summer conditions based on sample analyses in August 1976.

Algae were counted as total organisms per milliliter. These values give only an approximation of algal biomass, since the average size of the organisms can vary widely with species variation. More recent river algae counts have included identification to the genus level and an estimate by the laboratory technician of the average dimensions characterizing each genus. A fair estimate of biomass can be derived from this data. The writers believe this method of estimating biomass is more accurate than estimating the mass from chlorophyll A concentrations.

Evaluation of Reuse Feasibility:

The upper part of the Trinity River receives wastewater effluent from Dallas, Fort Worth, and other area cities. Except during brief periods following large rains, the river is dominated by the effluent flow. At the time (1976) of model calibration, the wastewater plants were discharging poor quality effluent. The effluent concentrations reached values as high as 83 mg/l for biochemical oxygen demand, and 88 mg/l for total suspended solids. These plants have been improved since that time, and as upgrading continues, the reuse of river water becomes more feasible.

A possible river water reuse scheme was analyzed using the model as calibrated for the Trinity River. Approximately half of the dry weather flow in the river 4 miles below the Dallas Central Wastewater

Plant would be pumped to the Elm Fork of the Trinity below Lake Lewisville (see Figure 3). From this discharge point, the water would flow downstream approximately 10 miles to the Dallas Elm Fork Purification Plant. At this point, about 2/3 of the water would be withdrawn by this plant, with the remaining flow continuing down the natural river channel 14 miles to a smaller purification plant, Dallas Bachman Plant. These two plants are presently supplied by releases into the river channel from Lake Lewisville and by the smaller Lake Grapevine. In the reuse evaluation simulation, the Lake Lewisville release was reduced to the amount needed to supplement the recycled water to meet the demands of the purification plants. Summer conditions were simulated, with recycled water making up 56 percent of the Elm Fork river flow.

Simulation Results

Algae concentrations of approximately 11,000 to 17,000 algae per milliliter were simulated near the purification plant raw water intakes. Algae concentrations in this range should not be an insurmountable problem. Algae data from Lake Ray Hubbard show that raw water with 18,000 algae per milliliter has been successfully purified. Water usage was curtailed after the algal counts (dominated by blue-green Oscillatoria) exceeded 20,000 cells/ml. Algae counts in excess of 20,000 cells/ml could probably be tolerated in a raw water supply if the predominant algae genera were not blue-greens.

Algae have been identified and enumerated in samples from two river sites since June 1978. One site, approximately 4 miles above the Dallas Central Wastewater Plant on the main stem of the Trinity River, receives flow enriched by upstream wastewater plants. The other site is approximately 20 river miles downstream from the Dallas Central Wastewater plant. Significant concentrations of blue-green algae were not found at either sample site during the summer and fall periods of sampling. Future sampling is needed to confirm that this summer's samples represent normal conditions. A representative summer low flow sample analysis is shown in Table 2 for the site above the Dallas Central Wastewater Plant.

Biochemical oxygen demand concentrations were simulated and used as an indication of organic content. Total organic carbon would probably have been a better indicator; and the explicit modeling of TOC is an improvement which would make the model more useful for studying water reuse feasibility. Simulated five day BOD concentrations were about 2 mg/l. Although Dallas does not have BOD data on raw water for comparison, the simulated concentrations are believed to be little higher than concentrations in some raw water currently being utilized.

Phosphate concentrations are also a concern to raw water quality. The Dallas purification plants use a lime-ferric sulfate process. The efficiency of solids removal by coagulation and precipitation may be impaired at phosphate concentrations in excess of 1 mg/l. A concentration of 0.9 mg/l phosphate was simulated near the raw water intakes.

Other possibly hazardous contaminants include metals, pathogenic bacteria, and viruses. Since virus populations were not monitored, and the coliform algorithms failed to explain the observed population variations during model calibration, the routines contained in the model for simulation of coliforms and viruses were not used in this

study. The effects of significant concentrations of possibly hazardous metals should also be analyzed prior to initiation of a water reuse project.

Discussion:

A large water supply system such as the Dallas system would normally have some flexibility in source utilization. This flexibility allows utilization of some sources on a seasonal basis. When algal growth is a problem, as in the case of late summer blue-green algae blooms, seasonal source utilization can be advantageous.

Recycled Trinity River water could probably be utilized (after wastewater treatment plant upgrading is completed) during late fall, winter, and early spring, without significant algae problems. This utilization would conserve water storage in the water supply reservoirs for use during the summer and early fall season.

A problem might arise during periods of heavy runoff into the Elm Fork. During recycling, benthic organic buildup would occur at a faster than normal rate as a result of circulating water with higher organic content. When heavy rains provided enough inflow, sediments would be resuspended, causing a rapid deterioration in water quality. In this case, Dallas could not totally curtail use of both Elm Fork purification plants. The plants could be reduced to minimum rates (one pump running at each plant) for the period of poor quality water (one to two days). Even if the water quality were somewhat degraded, and treatment costs increased, for short periods of time, the recycle operation would be less expensive than building additional distant water supply reservoirs; and it would be environmentally comparatively benign.

In summary, algae simulation indicates that recycling of Trinity River water, at least on a seasonal basis, would not create serious taste and odor problems. Other possible ill effects of such an operation would require further study.

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- (2) Lake Ray Hubbard Eutrophication Study, Volume III - Simulation Modeling of Phytoplankton. Dallas Water Utilities and the University of Texas at Dallas. Prepared for the North Central Texas Council of Governments. Dallas, Texas. (April, 1978).
- (3) Thomann, Robert V., Dominic M. DiToro, Richard P. Winfield, and Donald J. O'Connor. Mathematical Modeling of Phytoplankton in Lake Ontario prepared for the Environmental Protection Agency, Office of Research and Development. EPA-660/3-75-005. (March, 1975).

TABLE 1
TREATED EFFLUENT NUTRIENT CONCENTRATIONS AND
RESULTING MAXIMUM CHLOROPHYLL A CONCENTRATIONS

SIMULATION DESCRIPTION		ORGANIC N mg/l	NH ₃ mg/l	NO ₃ mg/l	ORGANIC P mg/l	PO ₄ mg/l	MAXIMUM CHLOR A
							mg/l
1978 Treatment	5 mgd	4.4	19.5	1.0	1.0	1.7	23.5
	20 mgd	4.4	19.5	1.0	1.0	1.7	56.6
10-10-8 Treatment	5 mgd	3.5	8.0	6.0	0.5	1.5	20.6
	20 mgd	3.5	8.0	6.0	0.5	1.5	42.8

TABLE 2
DALLAS WATER UTILITIES
WATER RECLAMATION RESEARCH CENTER
ALGAE COUNTS/IDENTIFICATION
(ORGANISMS PER MILLILITER)

Sample Location Trinity River at Corinth Bridge Date 08-22-78
Sample Volume 100 ml Concentrate Volume 10 ml Magnification 200

DIATOMS		GREEN	
Amphora	163	Agmenellum	163
Cyclotella	490	Ankistrodesmus	1,469
Diatoma	816	Chlorella	2,611
Cymbella	490	Oocystis	82
Fragilaria	734	Pediastrum	408
Gomphonoma	163	Scenedesmus	1,714
Gyrosigma	163	Tetraedon	408
Navicula	1,387	Phytoconis	326
Synedra	653	Spirulina	245
Nitzschia	326	Selenastrum	1,387
Unknown	245	Unknown	2,611
TOTAL	5,630	TOTAL	11,424
FLAGELLATES		BLUE-GREENS	
Chlamydomonas	1,224	Oscillatoria	163
Euglena	898		
Phacus	898		
Unknown	898		
TOTAL	3,918		

Total Organisms/ml 21,135

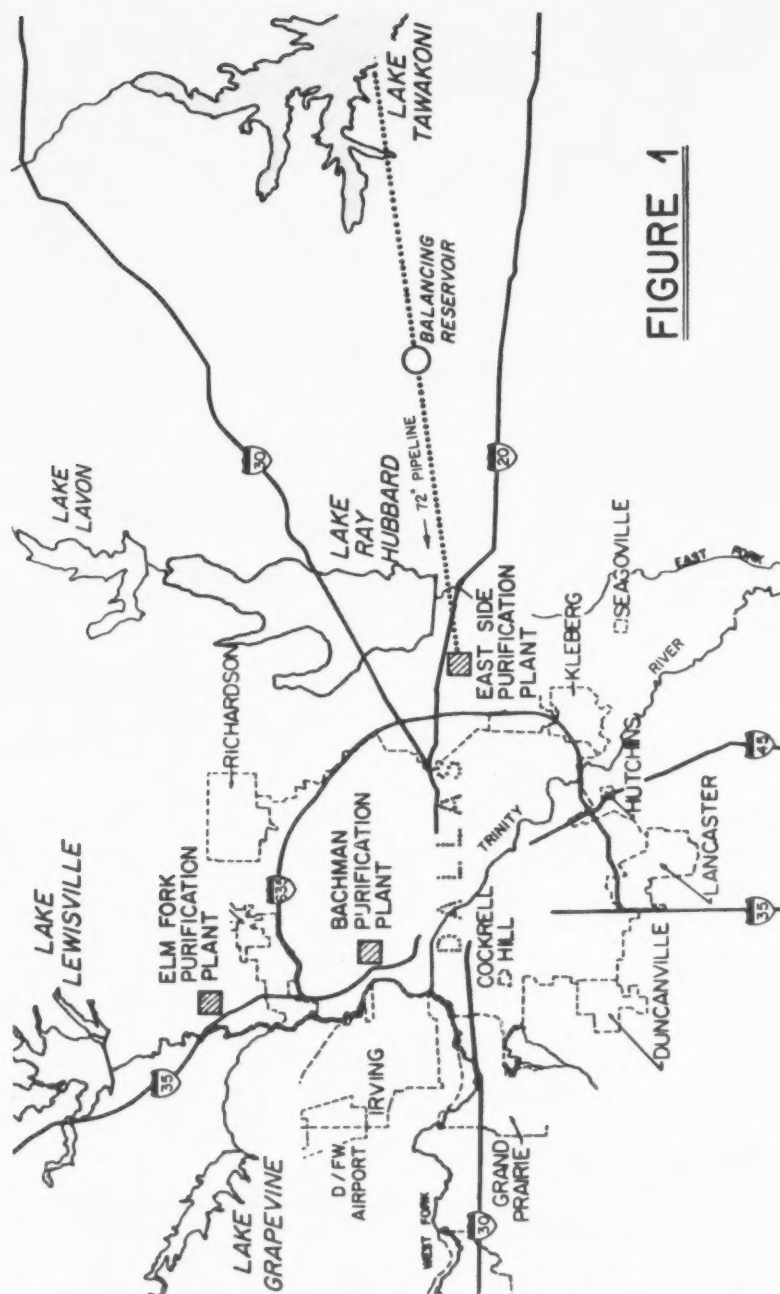
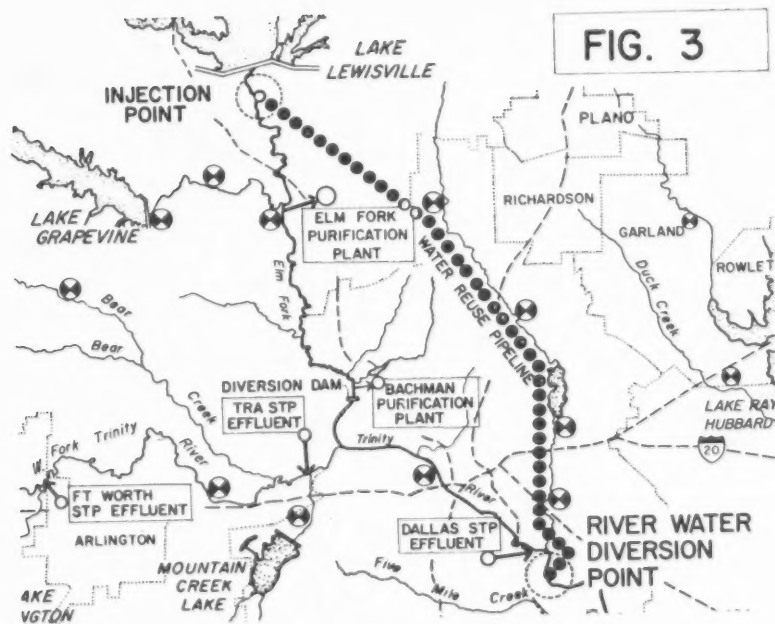
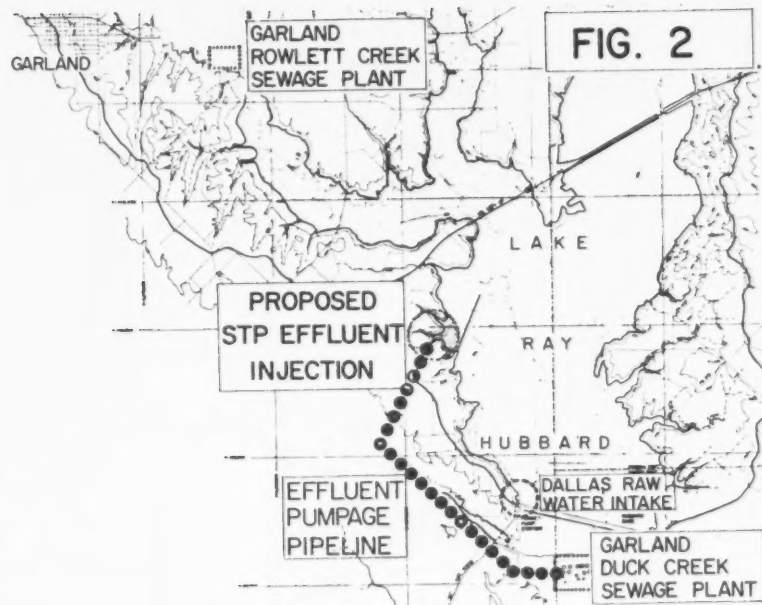


FIGURE 1



MONITORING ORGANIC LOADING WITH THE
ELECTROCHEMICAL ORGANIC CONTENT ANALYZER

William F. Cowen
Research Chemist
Catalytic, Inc.
Centre Square West
1500 Market Street
Philadelphia, PA 19102

Barry W. Peterman
Research Area Manager for Advanced Wastewater Treatment
U.S. Army Medical Bioengineering Research and Development Laboratory
Ft. Detrick, Frederick, MD 21701

Ronald J. Davenport
Program Manager
Life Systems, Inc.
24755 Highpoint Road
Cleveland, OH 44122

The need for simple, automated and low-cost monitors of organic loading will increase as water reuse technology becomes commonplace. These monitors will be used for monitoring and controlling the unit processes selected for organic removal. Monitors will be employed in some cases to monitor influents to the processes for feed forward control or for detecting influents that may foul process components. Other applications will call for organic monitors in process effluents for feedback control and for detection of component failures. In all cases it will be important to utilize simple, automated monitors to reduce operator training, labor costs and operator errors. Low hardware costs will permit these monitors to be used in smaller-scale applications in which more expensive instrumentation would not be cost-effective.

The Electrochemical Organic Content (EOC) Analyzer has been developed to satisfy this need. It monitors organic loading using differential capacitance measurements of adsorbed organic solutes on the surface of an indicating electrode. This technique has the advantages of automated operation, low cost and sensitivity to many organic solutes at concentrations of 1 mg/l total organic carbon (TOC) or less. The response of the EOC Analyzer is a general indicator of organic loading and is analogous to the response conductivity monitors exhibit for ionic solutes.

A breadboard version of the EOC Analyzer was developed by Life Systems, Inc. of Cleveland, OH, under contract to the U.S. Army Medical Bioengineering Research and Development Laboratory (USAMBRDL), Ft. Detrick, Frederick, MD under Contract DAMD-17-75-C-5070.⁽¹⁾ The initial application for the EOC Analyzer was for use in the Water Processing Element (WPE) Pilot Plant for the Medical Unit, Self-contained, Transportable (MUST) Hospital. More recently, USAMBRDL has been evaluating the EOC Analyzer in monitoring treatment of shower wastewaters for reuse using microfiltration (MF), ultrafiltration (UF) and reverse osmosis (RO). This paper reports the evaluation of the Breadboard EOC Analyzer for monitoring RO treatment of shower wastewaters.

MATERIALS AND METHODS

Breadboard EOC Analyzer

A detailed description of the Breadboard EOC Analyzer was reported previously.⁽¹⁾ It was designed for automated, on-line operation and is shown in Figure 1. Its mechanical schematic is presented in Figure 2. The electrochemical cell indicated in Figure 2 is the organic sensor and contains the indicating electrode on which organics adsorb. Electrolyte is added to the sample to reduce the solution resistance in the cell. The sample pump draws sample into the analyzer at a rate of 8 ml/min, and the electrolyte is injected into the sample at a rate of 2 ml/min. The temperature controller operates with feedback control from the temperature sensor downstream of the cell and maintains the cell at 25 C.

The effects of dissolved O_2 on the EOC measurement are eliminated by in situ electrochemical reduction of O_2 at the indicating electrode:



Electrochemical reduction of O_2 occurs when the flow of the sample/electrolyte mixture is bypassed around the cell by the solenoid valve. When the O_2 is eliminated from the cell, the EOC measurement is made and the solenoid valve directs a fresh portion of the sample/electrolyte mixture into the cell. This sequence and the sequence used for automatic, electrochemical pretreatment of the indicating electrode have been reported previously.⁽¹⁾

The ratio of sample and electrolyte flow rates should remain constant for most accurate EOC measurements. It was found, however, that the organics used during this test program affected the flow rate of the sample pump, and therefore caused the sample/electrolyte ratio to change. The sample pump is a piston pump which utilizes close tolerances between the piston and cylinder wall to alternately seal the pump inlet and outlet. The flow rate change was attributed to changes in the surface tension of the water because of the organic solutes. Some data was obtained by manually mixing electrolyte and sample prior to analysis. Other tests were performed using a dual-channel peristaltic pump to mix electrolyte and sample. Both procedures resulted in accurate EOC measurements.

Reagents

EOC Electrolyte. The EOC electrolyte was prepared by dissolving 610 g of reagent grade NaOH in 800 ml of distilled water. The solution was cooled and diluted to one liter and the specific gravity of the solution was measured to ensure that it was within the range of 1.43 to 1.46. A second solution consisting of 0.988 g of NaCl in one liter of water was prepared and combined with the NaOH solution to produce two liters of the electrolyte stock solution. Then 200 ml of the stock solution was diluted to one liter to prepare the electrolyte used in the analyzer.

Hyamine 3500. A stock solution of Hyamine 3500 was prepared by dissolving 2.52 g of the commercial preparation (supplied by Rohm & Haas, Co.) in distilled water and diluting to one liter. The commercial preparation consists of 20% ethanol and a mixture of homologues of N-alkyldimethylbenzylammonium chlorides. The homologue mixture consists of 50% C₁₄, 40% C₁₂ and 10% C₁₆.

Other Reagents. Stock solutions of other organic solutes were prepared at concentrations of 100 mg/l. (All concentrations are reported on a weight basis.) Dial hand soap bars were ground and a suspension of the soap in distilled water was prepared. Sodium lauryl sulfate (sodium dodecylsulfate) from Aldrich Chemical Co. was used. Carbitol (diethylene glycol monoethyl ether) and all other organics were obtained from Fisher Scientific Co.

TOC Measurements

Total organic carbon measurements of wastewaters were made with the Beckman Model 915 Carbon Analyzer. For most solutions with concentrations less than 10 mg/l TOC, the Dohrmann DC-54 Carbon Analyzer with a PR-1 sample input module was used.

Test Procedures

Sensitivity Tests. The response of the EOC Analyzer to individual organic solutes was quantified. Samples and electrolyte were mixed manually, and the response of the analyzer was referenced to a distilled water blank.

Long-Term Stability Test. Synthetic shower wastewater was prepared having the composition listed in Table 1. This water was treated in a RO unit process to produce sufficient effluent to operate the EOC Analyzer for 225 hours. Batches of the effluent were separated in 16 liter volumes and 8 liters of electrolyte were added to each batch. The EOC Analyzer sampled each batch continuously for 33 hours. The TOC concentration and conductivity of the sample/electrolyte mixture exiting the EOC Analyzer were measured as a check on the preparation of each batch.

Sensitivity Stability Test. The possibility of fouling the indicating electrode in the EOC cell during long-term exposure to wastewater was investigated. Fouling, if it occurred, would result in decreased response to organics. This was investigated by preparing a sample of the RO effluent used in the Long-Term Stability Test, above, and spiking it with Hyamine 3500 at a concentration of 5 mg/l. The response of the analyzer to this solution was measured by alternately injecting into the analyzer a sample of the original RO effluent and a sample of spiked effluent. Thirteen sets of these samples were used.

This procedure was carried out immediately before and after the Long-Term Stability Test. Therefore, any fouling would be detected as a difference in the sensitivity of the EOC Analyzer response to the Hyamine in the spiked samples before and after the 225 hour exposure to the wastewater.

Reproducibility Test. The speed and completeness with which the EOC Analyzer recovers from exposure to high organic concentrations was investigated in the Reproducibility Test. Samples of the RO effluent (designated F) and effluent samples diluted with three volumes of distilled water (designated D) were alternately injected into the analyzer. Each set of solutions was separated by distilled water samples, W. Each set was carried out in the order; W, D₁, F, D₂, W. Twelve sets were analyzed.

Response Interactions Test. The response characteristics of the EOC Analyzer to samples containing mixtures of organic solutes was investigated in this test. The EOC response to samples having known concen-

trations of Hyamine 3500 and Carbitol was quantified in a 3 x 3 factorial experiment. Nine solutions were prepared having all possible combinations of Hyamine at concentrations of 0, 1 and 5 mg/l and Carbitol at concentrations of 0, 1 and 10 mg/l. The response of the EOC Analyzer to these solutions was measured in duplicate experiments and referenced to distilled water blanks.

RESULTS AND DISCUSSION

The EOC Analyzer is based on a unique method of monitoring organic solute concentrations. The measurement is made by first adsorbing organics on the surface of the indicating electrode in the EOC cell. The capacitance decrease at the electrode/solution interface, resulting from the organic adsorption, is then measured and related to the organic concentration. A capacitance exists at the interface because a charge exists on the electrode and there is a layer of oppositely-charged ions adjacent to the electrode. In the absence of organics, the ions are located close to the electrode or are separated by a layer of water molecules. In the presence of adsorbed organics, the capacitance is less because the organics separate the ions from the electrode surface. The dielectric constant between the ions and the electrode also is decreased because water molecules are replaced by adsorbed organics. These two phenomena result in a capacitance decrease which is related to the fraction of the electrode covered by the adsorbed organic. Since the fractional coverage is basically a logarithmic function of organic concentration, the response of the EOC Analyzer is also a logarithmic function of concentration.

The Breadboard EOC Analyzer produces a digital output having instrument units that are proportional to the differential capacitance of the indicating electrode. The EOC value therefore decreases as organic loading increases.

The EOC Analyzer is analogous to a conductivity monitor. Conductivity monitors respond to ionic solutes with greater sensitivity to some ions than to others (Figure 3). For solutions containing solute mixtures, conductivity monitors provide a general indication of their ionic content. Because of the low cost, high reliability and automation of on-line conductivity monitors, they are frequently used for monitoring water quality and for process monitoring and control.

The Breadboard EOC Analyzer is an automated, on-line monitor of the general organic content of water and wastewater. Because of its inherent reliability and the low cost projected for end-item units, the EOC Analyzer is expected to satisfy many requirements for measuring organic loading in process monitoring and control applications.

Sensitivity Tests

The response of the Breadboard EOC Analyzer to four compounds that are constituents of shower wastes were quantified in order to determine the detection limit and range of response that the analyzer could be expected to exhibit when monitoring treated wastewaters. The four compounds were selected as representatives of cationic, nonionic and anionic synthetic detergents and soaps based on fatty acids. These compounds are listed in Table 2 as ingredients of shower wastes constituents. Hyamine 3500 (cationic), Carbitol (nonionic), sodium dodecylsulfate (anionic) and Dial hand soap were selected for this test.

The response curves for the Breadboard EOC Analyzer are shown in Figure 4. Besides the response of the EOC Analyzer to the four organic compounds listed above, previously obtained response curves for methanol and urea are included for comparison since these organics may also be contained in shower wastewaters.

The response of the EOC Analyzer to these organic solutes ranges from very sensitive response to Hyamine 3500 to negligible response to sodium dodecylsulfate at a concentration of 10 mg/l. Detection limits less than 1 mg/l were found for Hyamine, Carbitol and Dial hand soap. The response of the EOC Analyzer to larger concentrations of Hyamine was greater than for any of the other organic solutes. This is fortuitous since the toxicity of Hyamine 3500 makes it important that it be removed from the wastewater in the treatment process. Therefore, a monitor that is selectively sensitive for Hyamine or other toxic materials in the wastewater can be used to advantage in controlling the treatment process for the elimination of those toxic materials.

Long-Term Stability Test

The indicating electrode in the EOC cell is not fouled by long-term exposure to shower wastewaters of at least 225 hours. Figure 5 shows the results of the long-term test during which the EOC Analyzer was continuously exposed to RO-treated synthetic shower wastewater. The TOC concentration in the sample/electrolyte mixture exiting the analyzer is also shown. The EOC response varied only between 246 and 258 EOC units between the 8th and 154th hours. Following that, the EOC response dropped abruptly but this change was accompanied by a decrease in the conductivity of the sample/electrolyte mixture and low TOC values. It is apparent that the sample/electrolyte mixture used during this period of lower EOC values was not identical to the earlier batches. The last batch of solution injected into the analyzer at the end of the test resulted in the original EOC values and conductivity. This shows that when the sample/electrolyte mixture is consistent, the analyzer response varies by only 12 EOC units. By investigating the response of the EOC Analyzer to diluted samples of the treated wastewater and by measuring the TOC concentrations of those solutions it was found that the variation in EOC response of 12 units corresponds to an uncertainty in the TOC of only 0.8 mg/l.

Sensitivity Stability Test

The EOC Analyzer was challenged with a series of RO-treated wastewater samples (some spiked with 5 mg/l of Hyamine 3500 and others unspiked) before and after the Long-Term Stability Test described above. The two Sensitivity Tests were performed 10 days apart. The results of the tests showed that the EOC response change to the Hyamine 3500 in the spiked samples was approximately 70 EOC units both before and after the Long-Term Stability Test. This test also showed that Hyamine 3500 adsorbed very strongly in the analyzer and required approximately 30 minutes to be completely flushed from the analyzer after each injection. Recovery from exposure to other organic solutes has never been observed to take as long; therefore, Hyamine 3500 represents a worst case test of the EOC performance. Since fouling of the indicating electrode and memory effects in the EOC response were not observed with Hyamine 3500, they are unlikely to be encountered with more common organic solutes which do not adsorb as strongly.

Reproducibility Test

The response of the EOC Analyzer to alternated samples of distilled water (W), diluted (D) and undiluted (F) RO-treated shower wastewater is shown in Table 3. This data was statistically evaluated. Any difference between the response to the first (D_1) and second (D_2) diluted samples in each run is an indication of carryover of organics and incomplete recovery from the organics in previous samples.

The data was statistically evaluated with a paired t test on the D_1 - D_2 pair, with a null hypothesis that $D_1 = D_2$. Any difference between D_1 and D_2 is an indication of carryover. The results in Table 3 indicate that within a 95% confidence level no significant carryover of organics from prior samples occurs.

Response Interactions Test

The response interactions from the replicated matrix of Carbitol and Hyamine 3500 solutions were tested statistically by analysis of variants, using the data in Table 4. The S-ratio for Hyamine-Carbitol interaction was 1.85, which was not significant at the 21% level of confidence. Consequently, response interactions were not serious and the EOC Analyzer apparently responds in an additive manner to each chemical in the presence of the other.

SUMMARY AND CONCLUSIONS

The Breadboard EOC Analyzer demonstrates that measurements of organic loading based on the differential capacitance technique can be automated for process monitoring and control applications. The response of the EOC Analyzer is a general indicator of organic loading and varies in sensitivity depending upon the composition of the organic solutes in the wastewater. In the test program reported here, the greatest response was produced by Hyamine 3500. This is particularly fortuitous since Hyamine is a toxic material and any treatment process designed for purification of shower wastewaters should be controlled so as to ensure elimination of toxic materials such as Hyamine 3500.

Other tests showed the EOC Analyzer is not fouled by long-term exposure to the simulated shower wastewater. The EOC Analyzer did not exhibit memory effects, and its response to solutions containing mixtures of Carbitol and Hyamine 3500 was essentially an additive function of the concentration of the two compounds.

A minor mechanical problem was noted during the testing program when the sample pump flow rate was found to be influenced by the presence of organics in the sample. The flow rate change was attributed to surface tension changes in the sample/electrolyte mixture. The problem was solved by using a dual-channel peristaltic pump. With this mechanical refinement, the EOC Analyzer is expected to provide the low-cost method of monitoring organic loading that will be required in many water reuse process monitoring and control applications.

ACKNOWLEDGMENT

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USAMBRDL Project Officer for the Development Contract on the EOC Analyzer, in coordinating this effort with Life Systems, Inc., is also gratefully acknowledged.

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Table 1
Synthetic Shower Waste

	Chemical	Concentration, mg/l
1.	Shower/Lavatory Cleaner (SBS-52)	220
2.	Sodium Chloride	183
3.	Dial Soap	152
4.	Talc	44
5.	Soil (Kaolinite)	42
6.	Hair Shampoo	5
7.	DEET (Insect Repellent)*	2
8.	Urea	2

Table 2
Ingredients of Shower Waste Constituents

1. Shower/Lavatory Cleaner, (SBS-52)
 - Alkyl (C₁₄-C₁₆) Dimethylbenzylammoniumchloride
 - Tripoly Phosphate
 - Isomeric Linear Alcohols (C₁₁-C₁₉)
 - Polyethoxy Ethanol
 - Ethanol
 - Isobornyl Acetate
2. Shampoo (Giant Brand)
 - Sodium Myristyl Sulfate
 - Sodium Lauryl Sulfate
 - Sodium Chloride
 - Cocamide DEA
 - Polyethylene Glycol
 - 55-Lanolin
 - Formaldehyde
 - Citric Acid
 - D&C Green #5 and #8
 - Trisodium Hydroxy EDTA
3. DEET (Insect Repellent)*
 - N, N-Diethyl-m-Toluamide, 71.25%
 - Other Diethyl Toluamides, 3.75%
 - Denatured Ethanol, 25%
4. Formula of a Typical Hand Soap
 - Coconut Oil, 6.4%
 - Castor Oil, 1.1%
 - Cottonseed Oil, 13.9%
 - Caustic Soda, 3.2%
 - Caustic Potash, 7.5%
 - Sodium Carbonate, 0.2%
 - Sodium Chloride, 0.4%
 - Alcohol, 1.5%
 - Sugar, 1.1%
 - Water, 64.3%
 - Perfume, 0.3%

*FSN 6840-00-753-4963

Table 3
Reproducibility Test Data

Test Set	D ₁	F	D ₂
1	362	272	378
2	379	280	378
3	379	291	380
4	383	293	389
5	380	293	389
6	379	295	375
7	380	294	378
8	387	298	379
9	379	295	377
10	379	292	378
11	379	289	376
12	378	292	377

Table 4
Response Interactions Test Data

RESULTS

Sample No.	Concentration, mg/l		EOC Response	
	Carbitol	Hyamine	Run No. 1	Run No. 2
1	0	0	53	29
2	0	1	25	44
3	0	5	212	245
7	1	0	37	20
8	1	1	88	67
9	1	5	216	231
4	10	0	24	71
5	10	1	115	92
6	10	5	253	242

ANALYSIS OF VARIANCE

Source	DF	Sum of Squares	Mean Square	F ratio	P(<J Comp. F)
Total	17	138406			
Carbitol	2	3187	1593.50	4.39	0.052
Hyamine	2	129614	64807.16	178.65	0.000
Carbitol X Hyamine	4	2684	671.17	1.85	0.213
Error	8	2902	362.75		

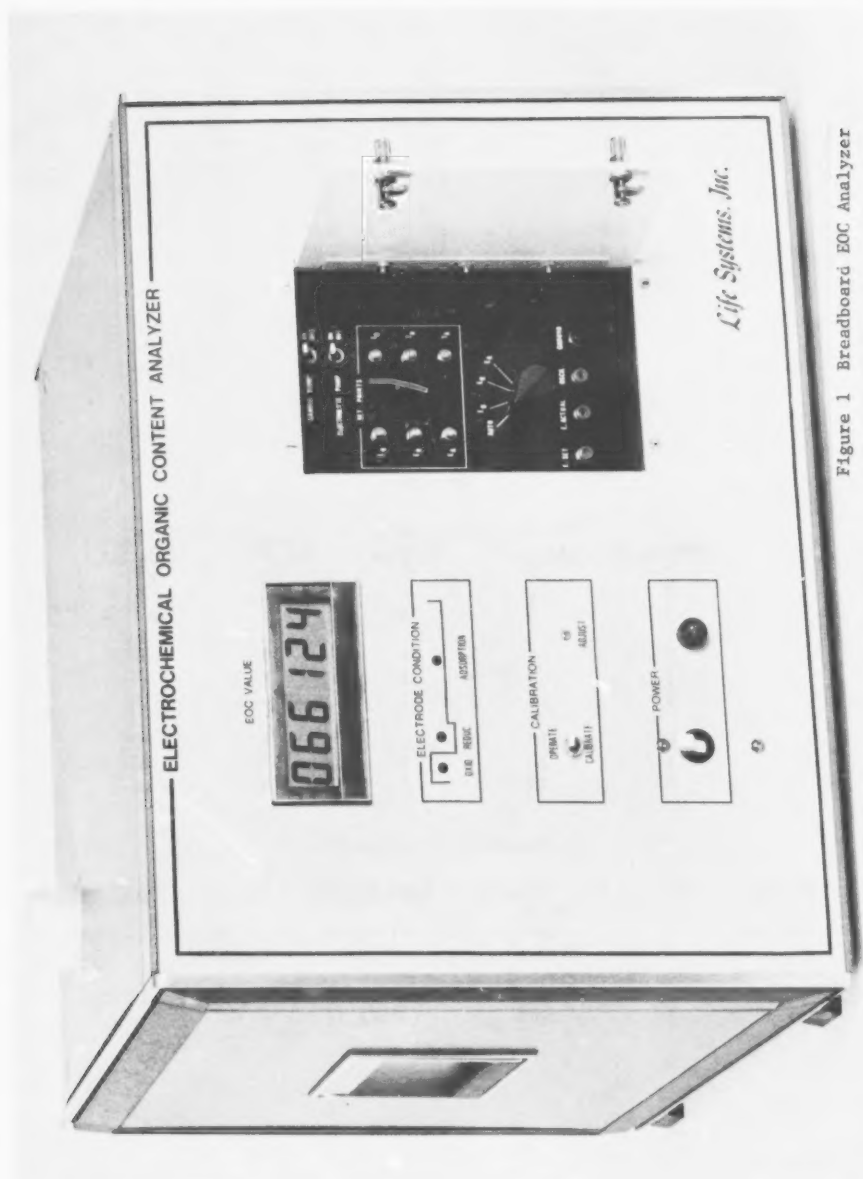


Figure 1 Breadboard EOC Analyzer

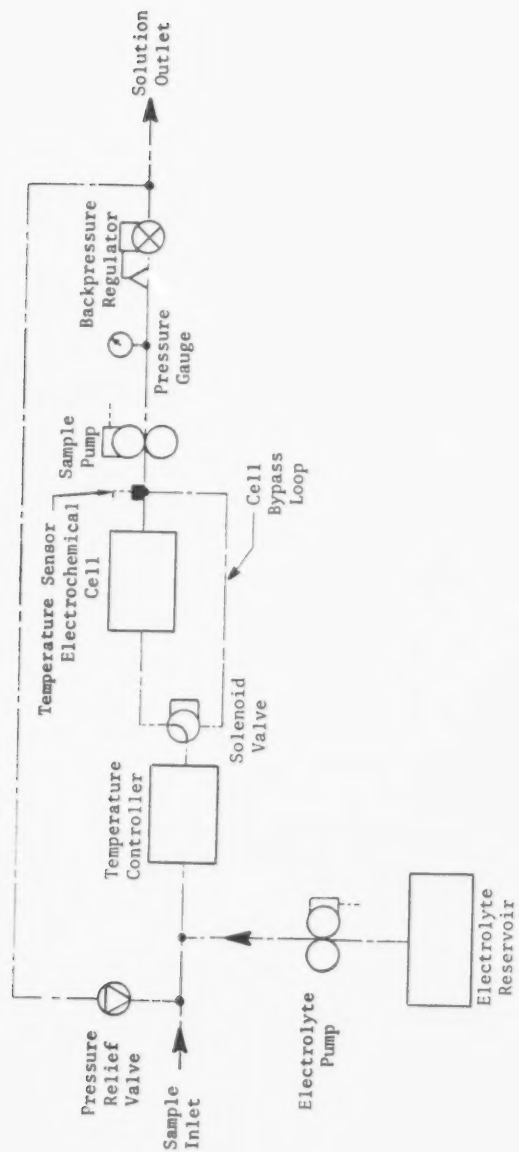


Figure 2 Mechanical Schematic of Breadboard EOC Analyzer

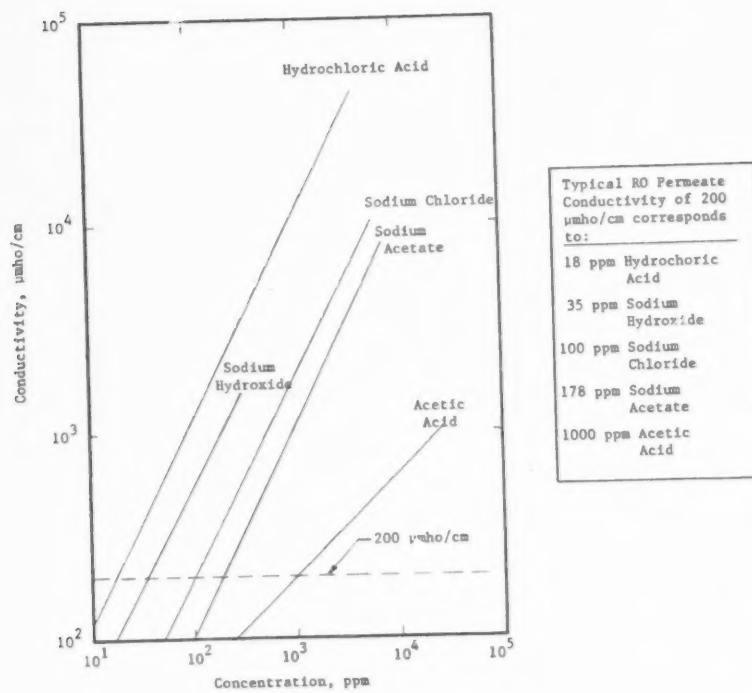


Figure 3 Conductivity Response Curves

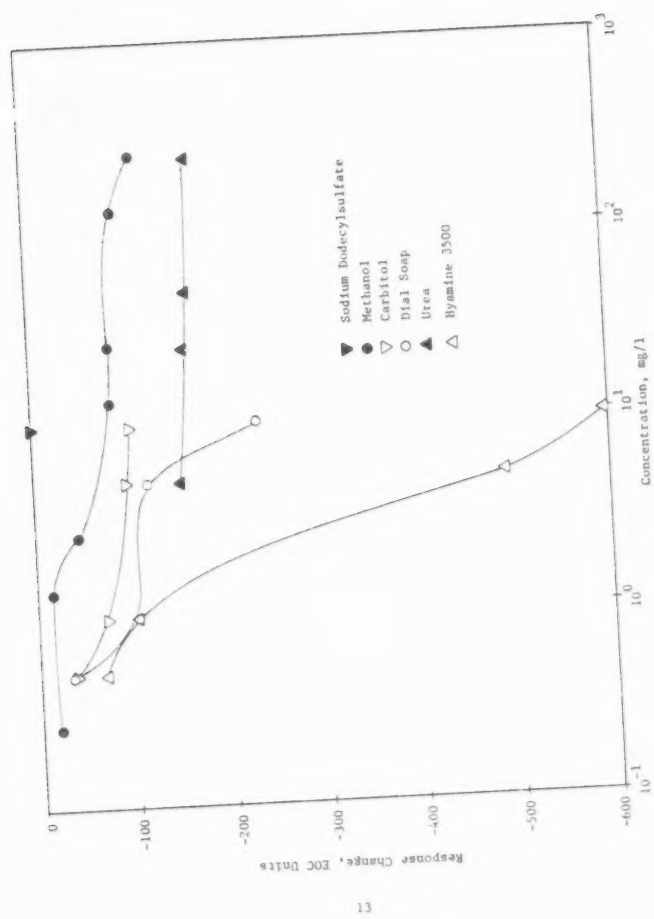


Figure 4. EDC Response Curves



Figure 5 Results of Long-Term Stability Study

MONITORING ORGANIC LOADING WITH THE ELECTROCHEMICAL ORGANIC CONTENT ANALYZER (ECOC)

B. J. Daveport

B. W. Peterman

W. I. Cowan



NO. 18

ABSTRACT

To evaluate the ECOC analyzer concept for monitoring treatment of shower wastewater for reuse.

BACKGROUND

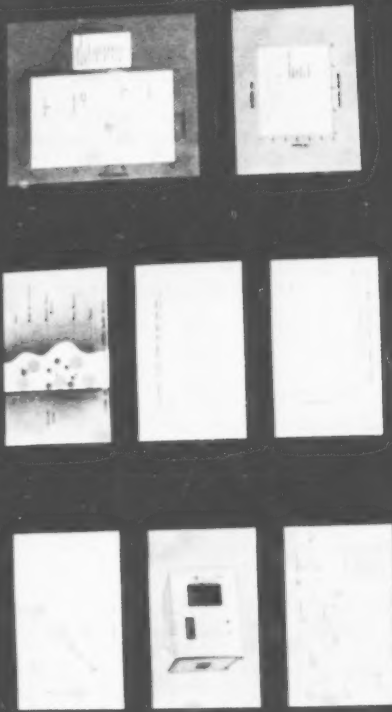
The ECOC analyzer was developed to provide a low-cost automated organic monitor for process monitoring and control applications. It provides a general response to organic solvents, alcohols, ketones, and esters in the presence of conductive, neutral to basic solvents.

EXPERIMENTAL

Major constituents of shower wastes were identified and the response of the ECOC analyzer to significant organics was quantified.

CONCLUSIONS

- 1) ECOC responds to the measurement of organic loading and area.
- 2) Sensitivity to organics is high.
- 3) ECOC analyzer is a feasible approach to low-cost, automated monitoring for treatment and reuse of shower wastewater.



DYNAMIC SIMULATION IN THE DESIGN AND ANALYSIS OF WATER REUSE UNITS

David M. Starks
Department of Chemical Engineering
Louisiana State University
Baton Rouge, Louisiana 70803

Cecil L. Smith
Department of Chemical Engineering
Louisiana State University
Baton Rouge, Louisiana 70803

Warren T. Abbott
Department of Chemical Engineering
Louisiana State University
Baton Rouge, Louisiana 70803

I. Introduction

Water reuse processes are being evaluated by the U.S. Army Biomedical Research and Development Command for a variety of applications, one example being a unit to support a field military hospital. The analysis of these processes necessitates the consideration of unsteady state behavior. Unit processes such as ultrafiltration and reverse osmosis are normally operated in a cyclical fashion, such as 20 hours on-stream and four hours regeneration. Furthermore, the flow rate and concentrations are often highly variable in the Army's potential applications. Size and weight limitations prohibit the incorporation of sufficient capacity to smooth these variations.

The experimental facility operated by the U.S. Army Biomedical Research and Development Command enables test data to be obtained regarding the performance of various unit processes on a particular waste water. From this test data, the parameters in a fundamental model of the unit process can be developed. The performance of a proposed design is then predicted based on these parameters.

To provide the capability to investigate a variety of process configurations and process operating strategies, a general purpose dynamic simulator package was developed. The structure of the steady state flow sheet simulators was chosen over that of the digital simulation languages that have evolved from analog simulators. The simulator is stream and equipment oriented, with the configuration and design parameters specified by input data.

II. Streams

For purposes of the simulator, the process is represented by individual pieces of equipment interconnected by process streams. Associated with each stream is a stream vector, which specifies the status of the stream.

The first element in the stream vector is always the flow rate. The remaining elements define the properties of the stream (composition, temperatures, etc.). As presently implemented for reuse processes, the stream vector consists of the following:

Element	Definition
1	Flow, m ³ /hr
2	Concentration of suspended solids, gm/m ³
3	Concentration of dissolved solids, gm/m ³
4	Concentration of total organic carbon, gm/m ³

The addition of temperature and/or pressure was originally envisioned, but has not been required for applications to date.

III. Equipment

For each piece of equipment, the following information must be specified:

1. Equipment number
2. Equipment type
3. Input streams
4. Output streams
5. Parameters specifying equipment characteristics

For each piece of equipment, a total of five streams may be specified, with any mix of input and output streams. By convention, input streams are designated by positive stream numbers and output streams by negative stream numbers. Normally all input streams appear prior to output streams. For certain types of equipment, the order of the streams is significant and the data parameters are specific to each type of equipment.

The simulator presently recognizes the following types of equipment:

Type	Code
Mixed tank	MT
Overflow tank	OT
Pump	P
Stream splitter	SP
Stream source	SO
Stream mixer	SM
Ultrafiltration unit	UF
Reverse Osmosis unit	RO
UV/ozonation unit	UV
Hypochlorination unit	HC
Sink	SK

As the reuse applications require, other pieces of equipment can be easily added to the simulator.

Several of the models for the more important pieces of equipment must also be used in a parameter fitting package to determine the values of model parameters from the experimental data. These include ultrafiltration, reverse osmosis, ozonation, and hypochlorination. To incorporate these models into the simulator package, special interfacing routines were written so that the same model subroutines could be used in the parameter fitting package and the simulator.

III.1 Mixed Tank

The mixed tank is described by a total material balance and three component material balances. The total material balance is (constant density assumed):

$$\frac{dV}{dt} = \sum_{\text{input}} F_i - \sum_{\text{output}} F_j$$

where V = volume of liquid in the tank, m^3
 F_i = flow rate of input stream, m^3/hr
 F_j = flow rate of output stream, m^3/hr
Each component material balance is:

$$\frac{d}{dt}(VC) = \sum_{\text{input}} F_i C_i - \sum_{\text{output}} F_j C_j$$

where C = concentration in the tank, gm/m^3
 C_i = concentration of the input stream, gm/m^3
 C_j = concentration of output stream, gm/m^3

Any combination of input and output streams are permitted. The concentration of all exit streams are the same as the concentration in the tank.

For the mixed tank, the following parameters are required:

- Par 1: Initial volume of liquid in the tank
- Par 2: Initial concentration of suspended solids in the tank
- Par 3: Initial concentration of dissolved solids in the tank
- Par 4: Initial concentration of TOC in the tank

III.2 Overflow Tank

The basic equations of the overflow tank are the same as for the mixed tank. However, the first stream must be the overflow stream and is specified as follows:

$$F_1 = F_T \text{ for } 0 < V < V_{\max}$$

$$F_1 = \sum_{\text{input}} F_i - \sum_{\text{output}} F_j \text{ for } V = 0 \text{ or } V = V_{\max}$$

where F_1 = overflow rate, m^3/hr

F_T = design overflow rate, m^3/hr

V = volume of liquid in tank, m^3

$V_{\max}$ = maximum volume of liquid in tank, m^3

F_i = flow rate of input stream, m^3/hr

F_j = flow rate of output stream, m^3/hr

The following six parameters are required for the overflow tank:

- Par 1: Initial volume
- Par 2: Initial concentration of suspended solids
- Par 3: Initial concentration of dissolved solids
- Par 4: Initial concentration of TOC
- Par 5: Design overflow rate, F_T
- Par 6: Maximum volume, $V_{\max}$

III.3 Volumetric Pump

This equipment represents either a constant volume pump or a variable volume pump followed by a flow controller. The first stream must be the input stream; the second must be the output stream.

The basic equation for this pump is as follows:

$$F_1 = F_2 = F_p$$

where F_p = flow rate through pump, m^3/hr

F_1 = flow rate of input stream, m^3/hr

F_2 = flow rate of output stream, m^3/hr

Furthermore, the concentration of the output stream is set equal to the concentration of the input stream.

The only input parameter is F_p .

III.4 Stream Splitter

The purpose of this piece of equipment is to split an input stream into two output streams, the flow rate of one output stream being specified. The streams are specified in the following order:

1. Input stream
2. Output stream whose flow is fixed

3. Output stream whose flow varies
The flow equation describing this piece of equipment is as follows:

$$F_2 = \begin{cases} F_s & \text{if } F_1 \geq F_s \\ F_1 & \text{if } F_1 < F_s \end{cases}$$

$$F_3 = F_1 - F_2$$

where F_1 = flow rate of input stream, m^3/hr

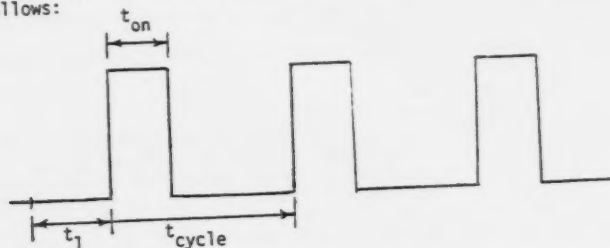
F_2 = flow rate of first output stream, m^3/hr

F_3 = flow rate of second output stream, m^3/hr

The concentrations in both output streams are set equal to the input concentration.
The only parameter required is F_s .

III.5 Stream Source

The stream source is designed to simulate the pulse-type waste streams that are typically encountered in units such as field hospitals. The stream is assumed to behave as follows:



where t_1 = time of start of first pulse, hr

t_{on} = time duration of pulse, hr

t_{cycle} = time of cycle, hr

During the flow period, the flow rate and concentration must be specified.

For this unit, the following parameters are required:

Par 1: Time of first pulse (t_1)

Par 2: Time duration of pulse (t_{on})

Par 3: Time of cycle (t_{cycle})

Par 4: Flow rate during pulse

Par 5: Concentration of suspended solids

Par 6: Concentration of dissolved solids

Par 7: Concentration of TOC

III.6 Stream Mixer

The stream mixer is designed to combine up to four input streams to produce a single output stream. In specifying the streams the output stream must be specified last.

The flow equations describing this unit are as follows:

$$F_j = \sum_{\text{input}} F_i$$

where F_i = flow rate of input stream, m^3/hr
 F_j = flow rate of output stream, m^3/hr

The equation for the respective concentrations is as follows:

$$C_j = \frac{\sum_{\text{input}} F_i C_i}{F_j}$$

where C_i = concentration of input stream, gm/m^3
 C_j = concentration of output stream, gm/m^3
No parameters are required for this unit.

III.7 Tubular Ultrafiltration Unit

Each ultrafiltration tube is modeled as a long hollow cylinder with a semi-permeable membrane adhering to the inner surface of the tube. The membrane acts as an extra fine mesh, retaining suspended solids and some very large organics, but passing water and much of the dissolved solids. During operation, a thin boundary layer is assumed to form at the membrane through which water must pass before reaching the tube wall. Detailed equations derived from writing component and overall material balances for the bulk and boundary layer sections of the tube form the basis of the model(1).

At the present, the model is concerned with three contaminants in the feed:

1. Suspended solids
2. Dissolved solids
3. TOC (Total Organic Carbon)

As written, the model assumes that the effect of suspended solids and dissolved solids on the flux of contaminants through the membrane is the same, permitting the dissolved and suspended solids to be lumped into a single contaminant, the total solids for the feed and concentrate streams. Since experimental evidence indicates that the membrane achieves essentially complete rejection of suspended solids, the permeate is assumed to be comprised of water, dissolved solids, and TOC.

In most configurations, the residence time of liquid in the UF tubes will be negligible in comparison with the dynamic contribution of associated equipment. Therefore, a steady state formulation has been used.

The streams are specified in the following order:

1. Permeate stream (output)
2. Concentrate stream (output)
3. Feed stream (input)

The ultrafiltration model predicts the composition and flow rate of the permeate stream. The status of the concentrate stream is calculated from total and component material balances formulated around the ultrafiltration unit knowing the conditions of the feed and permeate streams.

The required parameters (in order) are:

1. Number of tubes in the module
2. Operating temperature
3. Inlet pressure
4. Pressure loss down the length of the tube.

III.8 Reverse Osmosis Unit

The model equations developed characterize the operation of the hollow fiber reverse osmosis separator(2). Contaminated water enters via the high pressure feed tube. Flowing in the radial direction, contaminated water becomes progressively more concentrated as water preferentially permeates through the membrane wall into the inner bore of the fibers. Concentrate is collected at the flow screen and moves

axially towards the concentrate outlet. Purified water collected inside the fibers exits the module as permeate.

Using the same justification as in the case of ultrafiltration, the dynamics of the reverse osmosis module are assumed negligible in comparison to associated equipment.

The streams are specified in the following order:

1. Permeate stream (output)
2. Concentrate stream (output)
3. Feed stream (liquid)

The reverse osmosis model predicts the composition and flow rate of the permeate stream. The status of the concentrate stream is determined in a manner analogous to that for the ultrafiltration unit described above.

The required parameters (in order) are:

1. Operating pressure
2. Operating temperature

III.9 UV/Ozonation Unit

The ozone contactor chosen for post reverse osmosis treatment varies slightly from the design by Torricelli (3). The purpose of the contactor is to oxidize organic compounds to carbon dioxide using gaseous ozone.

During normal operation, contaminated water enters the pre-contactor, flows sequentially through each of six stages, and exits as purified water. Ozone dispersed in a carrier gas (usually air) is sparged through each of the stages, collected at the head space above each stage, and sparged through the pre-contactor. This flow path for the gas results in a high utilization of ozone, which is particularly important since the power requirements for ozone generation are usually high.

Turbulence produced by the bubbles of gas swirling upward through the column of liquid and the significant liquid residence time cause the aqueous phase of each stage to approach the perfectly mixed state. The bubbles are assumed to be of uniform size and shape, and each is considered to maintain a uniform velocity through the contactor.

Streams are specified in the following order:

1. Influent stream (input)
2. Effluent stream (output)

The required initial conditions and equipment parameters follow (in order):

1. Initial concentration of suspended solids.
2. Initial concentration of dissolved solids.
3. Initial concentration of total organic carbon.
4. Inlet gas concentration of ozone (moles O_3 /mole carrier gas).
5. Volumetric gas flow rate.
6. Code indicating the presence of the optional pre-contactor. (If the value specified is $\neq 0.0$, the pre-contactor is assumed present.)
7. Number of contactor stages (each contactor stage is assumed to be fitted with UV lamps).

The ozonation system model is concerned only with the concentration of TOC. The concentrations of dissolved and suspended solids are not changed by this unit.

III.10 Hypochlorination Unit

Ozone is a powerful disinfectant but is highly unstable in aqueous solutions. However, a small residual of free available chlorine, present as either $HOCl$ or OCl^- , is an effective retardant against bacterial growth for sustained periods. The purpose of the hypochlorination module is to introduce the proper amount of calcium hypochlorite solution to achieve the required disinfectant residual without an excessively high free chlorine concentration.

Streams are specified in the following order:

1. Influent stream (input)

2. Effluent stream (output)
The required initial conditions (in order) are:

1. Initial pH
2. Initial concentration of free available chlorine
3. Initial concentration of suspended solids
4. Initial concentration of dissolved solids
5. Initial concentration of total organic carbon

The hypochlorination system model is concerned only with pH and the concentration of free available chlorine. The concentration of dissolved solids, suspended solids and TOC are not changed by this unit.

III.11 Sink

As indicated earlier, all input streams to the process must originate with the source block. Similarly, all streams leaving the process must be inputs to a block called the "Sink". One reason for this is so that each stream must originate with one and only one block and the destination for each stream must be one and only one block.

Actually, the sink block plays no role in the simulation. The only calculation procedure involved is that the collective stream flows (total and component) are integrated for the material balance calculations.

Each sink block may have up to five input streams. The process configuration may contain multiple sink blocks.

IV. Control Package

This section serves to present the various elements which provide the capability of formulating a control strategy for any modeled process configuration.

1. Sensor elements
2. Control elements
3. Manipulator elements

IV.1 Sensor Element

A sensor element is used to simulate a variety of process indicator units such as a composition analyzer, a liquid level indicator, or a pH probe. In addition, the sensed value is assumed to depend upon actual value according to a first order lag relationship. That is,

$$\tau \frac{dC(t)}{dt} + C(t) = R(t)$$

where C = sensed value
R = actual value
 τ = time constant
t = time

If τ is extremely small in magnitude, C(t) approaches R(t) for all values of time.

The sensor element may be used to measure any element of a stream or any parameter for an item of equipment. Future plans are to incorporate a dead time into the sensor block so that certain analyzers can be properly incorporated.

IV.2 Controller Element

Several controller elements have been developed and are described subsequently. Each of these obtains process information via sensor elements and outputs control actions via manipulator elements.

IV.2.1 PID Controller

This element uses the PID algorithm (Proportional plus Integral plus Derivative) in the so called "velocity" form to manipulate a process variable given the measurement of a related process variable. The continuous formulation of the PID velocity algorithm is as follows:

$$\frac{dM(t)}{dt} = K_C \frac{dE(t)}{dt} + \frac{E(t)}{T_i} + T_d \frac{d^2E(t)}{dt^2}$$

where M = manipulated variable
 E = measured error
 R = setpoint for controlled variable
 C = measured value of controlled variable
 K_C = controller gain
 T_i = integral time
 T_d = derivative time
 t = time

The values specified for K_C , T_i and T_d are known as the controller settings.

The PID element also allows for the controller being in the manual mode. In this case, the controller output is set to some specified value. A manual mode is indicated by a positive value for the mode parameter; the automatic mode is specified by a non-positive value.

IV.2.2. Ratio Controller

The ratio controller employs a particularly simple feedforward strategy. The controller output is calculated as a ratio or constant times the input measurement. That is,

$$M = R * C$$

where M = controller output
 R = ratio or multiplication constant
 C = measurement (input to the controller)

If the controller is on manual, M is set to a specified value. The manual mode is indicated by a positive value for the mode parameter; the automatic mode by a non-positive value.

The required parameters (in order) are:

1. The identification number of the sensor for the controlled variable.
2. The identification number of the manipulator for the manipulated variable.
3. The ratio of multiplication constant.
4. The controller mode.
5. The manual setting - used only if parameter 4 is greater than zero.

Initialization calculations are the same as normal calculations.

IV.2.3 Binary (or On-Off) Controller

This controller is often used where tight control is unnecessary or undesirable. The algorithm employed is described below:

$$M = M_H \text{ if } C > C_H \text{ and } C > C_L$$

$$M = M_L \text{ if } C < C_H \text{ and } C < C_L$$

where M = controller output for manipulated variable
 C = sensed value of measured variable
 C_H = upper limit on tolerable region for C
 C_L = lower limit on tolerable region for C
 M_H = setting for M if C is above C_H and C_L
 M_L = setting for M if C is below C_L and C_H

If C is between C_L and C_H , M remains unchanged.

This controller also provides for a manual override feature. If the controller mode is positive, M is set to some specified value; otherwise, the controller is on automatic and M is determined by the technique outlined above.

IV.3 Manipulator Element

This element is used to set an equipment parameter to a desired value. If an attempt is made to specify the status of a stream vector, an error condition is noted and an appropriate message is written. That is, one may not arbitrarily change stream concentrations or flow rates.

The manipulator checks to see if an particular equipment setting is within specified limits -- limits which would normally correspond to extremes of physical reality.

The algorithm used follows:

$$V_o = V_H \text{ if } V_i > V_H$$

$$V_o = V_L \text{ if } V_i < V_L$$

$$V_o = V_i \text{ if } V_L \leq V_i \leq V_H$$

where V_o = output value from manipulator

V_i = input value to manipulator

V_H = upper limit on V_o

V_L = lower limit of V_o

V. Data Preparation

The input data cards provide for the entry of the following types of information:

1. Specification of the plant itself, including
 - a. Configuration (equipment and streams)
 - b. Design parameters
 - c. Initial conditions
2. Descriptions, including
 - a. Equipment names
 - b. Names for equipment parameters
 - c. Stream names
 - d. Names for stream elements
3. Run control parameters (time of run, integration interval, and print interval)
4. Variables to be printed, which may be either
 - a. Any element of any stream (e.g., flow rate of stream 5)
 - b. Any parameter of any item of equipment
5. Plot control information, including
 - a. Variables to be plotted
 - b. Time of plot
 - c. Arrangement of plots
6. Model parameters such as kinetic parameters, mass transfer coefficients, etc.

As the data is entered, checks are made to assure that the data is meaningful.

To assist in the preparation of a data check, data control cards are used to establish segments in the data deck. Table I lists the eight segments that establish the general layout of the data check. The following sections describe each segment in more detail.

If C is between C_L and C_H , M remains unchanged.

This controller also provides for a manual override feature. If the controller mode is positive, M is set to some specified value; otherwise, the controller is on automatic and M is determined by the technique outlined above.

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This element is used to set an equipment parameter to a desired value. If an attempt is made to specify the status of a stream vector, an error condition is noted and an appropriate message is written. That is, one may not arbitrarily change stream concentrations or flow rates.

The manipulator checks to see if an particular equipment setting is within specified limits -- limits which would normally correspond to extremes of physical reality.

The algorithm used follows:

$$V_o = V_H \text{ if } V_i > V_H$$

$$V_o = V_L \text{ if } V_i < V_L$$

$$V_o = V_i \text{ if } V_L \leq V_i \leq V_H$$

where V_o = output value from manipulator

V_i = input value to manipulator

V_H = upper limit on V_o

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As the data is entered, checks are made to assure that the data is meaningful.

To assist in the preparation of a data check, data control cards are used to establish segments in the data deck. Table I lists the eight segments that establish the general layout of the data check. The following sections describe each segment in more detail.

V.1 Model Parameters

For modules such as ultrafiltration (UF), reverse osmosis (RO), etc., the model parameters are determined by fitting the model to experimental data. Thus, it is necessary to provide a facility whereby these parameters can be easily changed.

V.2 Stream Elements

The purpose of this data segment is to read the engineering units and the descriptions of all elements of the stream vector.

V.3 Stream Names

The simulator provides for the association of a twenty-character descriptor with each process stream. The stream numbers may be entered in any order and it is not necessary that a descriptor be entered for each process stream. In the output generated from other sections of the simulator, the stream descriptor normally accompanies the stream number whenever sufficient space is available.

V.4 Configuration

For each item of equipment in the plant, two data cards are entered. The first card specifies the equipment number, equipment type, stream connections, and the descriptor for that item of equipment. The second card specifies the physical parameters for the respective item of equipment.

Upon completion of the entry of the plant configuration, a check is made to see that for each stream

- a. one and only one source is specified, and
- b. one and only one destination is specified.

When a duplicate source definition or a duplicate destination definition is encountered, messages are generated in the configuration output. If any errors are detected in either the configuration or in the parameters specified, the run is terminated.

V.5 Print Specifications

The simulator will generate a tabular listing of up to nine process variables as a function of time. The output may be specified to be either of the following:

1. Any element of any stream, or
2. Any parameter associated with any unit of equipment.

At least one print specification must be made.

V.6 Plot Specifications

The simulator will generate plots of up to five process variables as a function of time. The output may be specified to be either of the following:

1. Any element of any stream, or
2. Any parameter associated with any unit of equipment

If no plots are to be generated, the entire plot specification data segment is omitted.

The plots are not generated as the simulation is being executed, but instead the data is stored for plotting upon completion of the run. If more than five variables are to be plotted, the output is directed to a data file for subsequent plotting.

V.7 Run

The run segment of the input data consists of only one data card that contains the following information:

1. Total time for run
2. Integration step size
3. Print interval

All parameters are in hours.

Upon completion of the simulation, the simulator prints a material balance showing

1. Initial inventory
2. Amount input (from sources)
3. Final inventory
4. Amount output (to sink)

Due to round-off errors, the material balance does not generally close exactly (except for plants with small configurations). However, large errors would indicate a problem somewhere in the simulation.

V.8 Plot Control

As indicated in an earlier section, a maximum of five variables can be plotted. The simulator permits the output to be one plot containing all five variables, to be five plots containing one variable each, or to be any combination thereof. For each variable to be plotted, the plot character, zero-coordinate and maximum-coordinate must be specified.

VI. Example Application

One application of the simulation package was in the simulation of a full scale water treatment facility. The process configuration in Figures 1 through 5 is similar to the conceptual design of a water treatment unit to support field military hospitals.

Due to the varying flows of the waste streams, the volume in the equalization tank varies considerably during a 20 hour operating cycle. With the configuration in Figure 2, the concentrations also build up, as illustrated in Figure 6 for the suspended solids. These concentration buildups further cause the permeate flow from the ultrafiltration unit to decrease.

Another interesting observation is the steady degradation in the quality of the reverse osmosis (RO) composite permeate beginning soon after starting operation. Since the reverse osmosis concentrate is recycled back into the RO feed tank, concentrations in the tank tend to increase. This in turn results in a harder separation in the RO modules and higher concentrations in the permeate. The low purity RO permeate is dampened somewhat by the liquid volumes of the ozonation and hypochlorination systems, the final output being that of Figure 7.

For this particular simulation, the TOC levels of the ozonation system influent were much higher than those of the data points used to determine model parameters for this unit process. Although the ozonation system behavior was reasonable, the accuracy of the solution under these conditions must be questioned. Thus, it is not possible to completely separate the parameter fitting and the simulation functions, but instead they must be closely coordinated.

VII. Summary

The general purpose dynamic simulator has proven to be useful in analyzing the performance of proposed designs for water reuse applications. To be most effective, the simulation effort must be coordinated very closely with the experimental efforts required to characterize the performance of the unit processes on the waste stream of interest.

Literature Cited

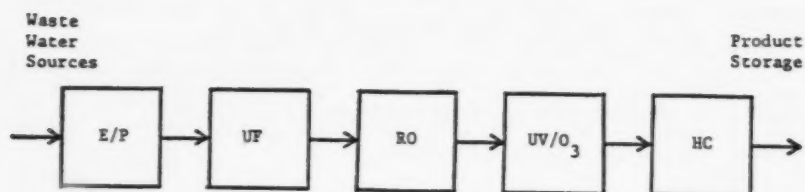
1. Starks, D. M., and C. L. Smith, "A Mathematical Model of a Ultrafiltration System for Water Purification Use", Proceeding of the 1978 Summer Computer Simulation Conference, Newport Beach, July 24-28, 1978.
2. Starks, D. M., and C. L. Smith, "A Mathematical Model of a Reverse Osmosis System for Water Purification Use", Proceedings of the Ninth Annual Pittsburgh Conference on Modeling and Simulation, Pittsburgh, April 27-28, 1978.
3. Torricelli, A., Advances in Chemistry, Series 21, p. 453, 1958.

Acknowledgement

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Table 1 Segment Definitions

<u>Segment</u>	<u>Segment Required/ Optional</u>	
Model Parameters	Optional	Used to read any model parameters required for modules such as UF, RO, etc.
Stream Elements	Required	Reads engineering units and description for each stream element
Stream Names	Optional	Reads description for each stream
Configuration	Required	Reads plant configuration
Print Definition	Required	Reads specifications for variables to be printed
Plot Definition	Optional	Reads specifications for variables to be plotted
Run	Required	Reads execution control parameters and initiates simulation
Plot Control	Required when plot used	Reads control information for generating plots



- E/P - Equalization/Pre-screening system
- UF - Ultrafiltration system
- RO - Reverse Osmosis system
- UV/O₃ - Ozone Contacting system
- HC - Hypochlorination system

Figure 1. Example Process Configuration

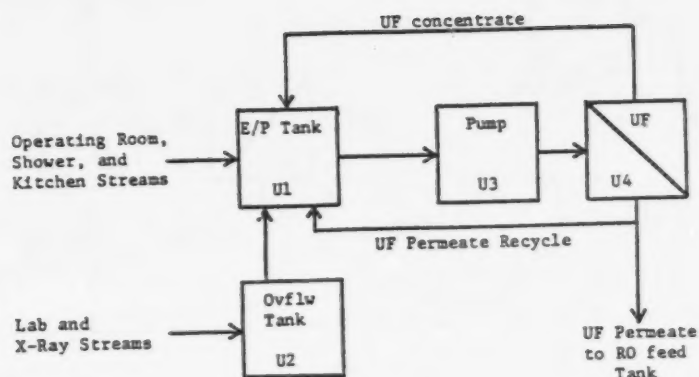


Figure 2. Expanded Equalization/Pre-screening and Ultrafiltration Systems

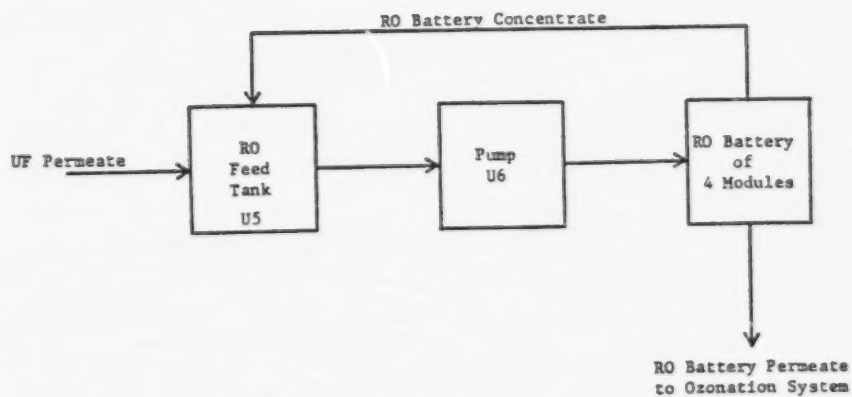


Figure 3. Expanded Reverse Osmosis System

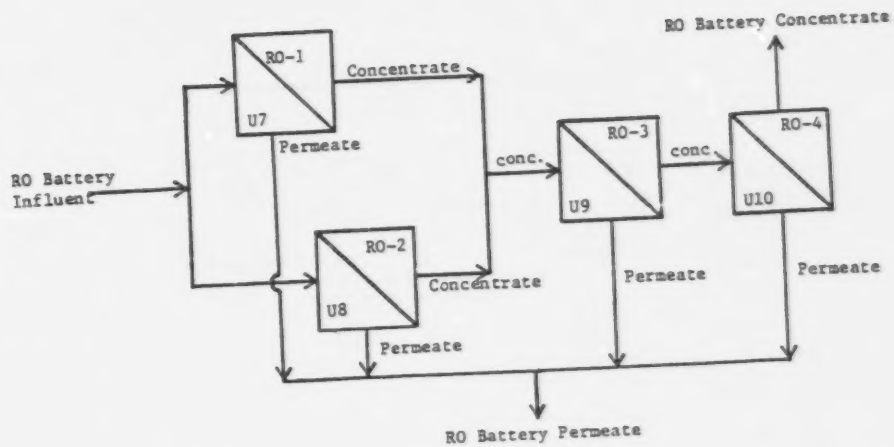


Figure 4. Expanded RO Module Battery Diagram

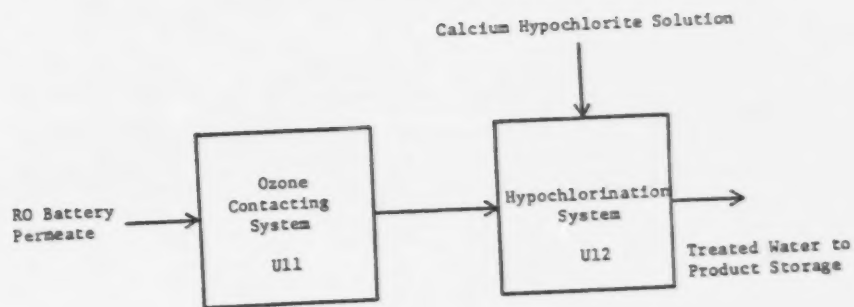
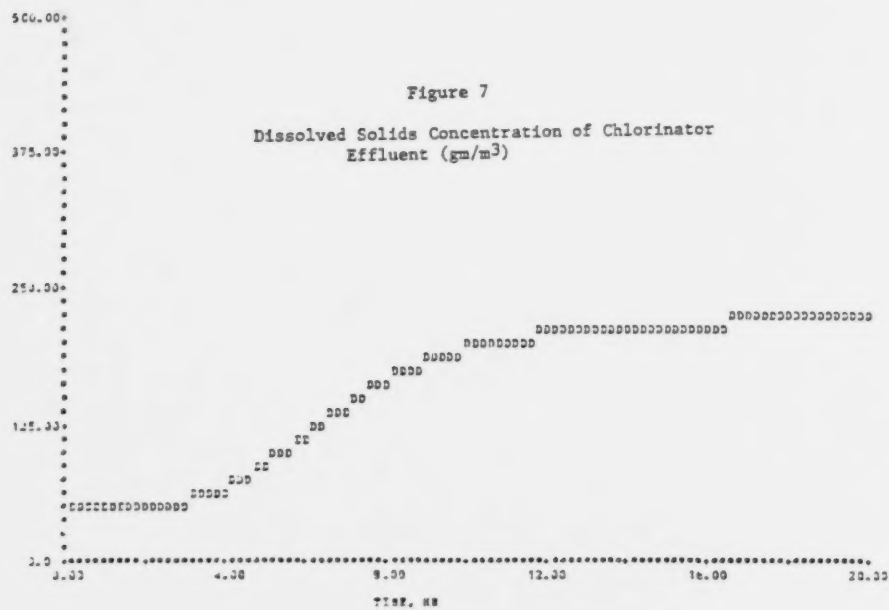
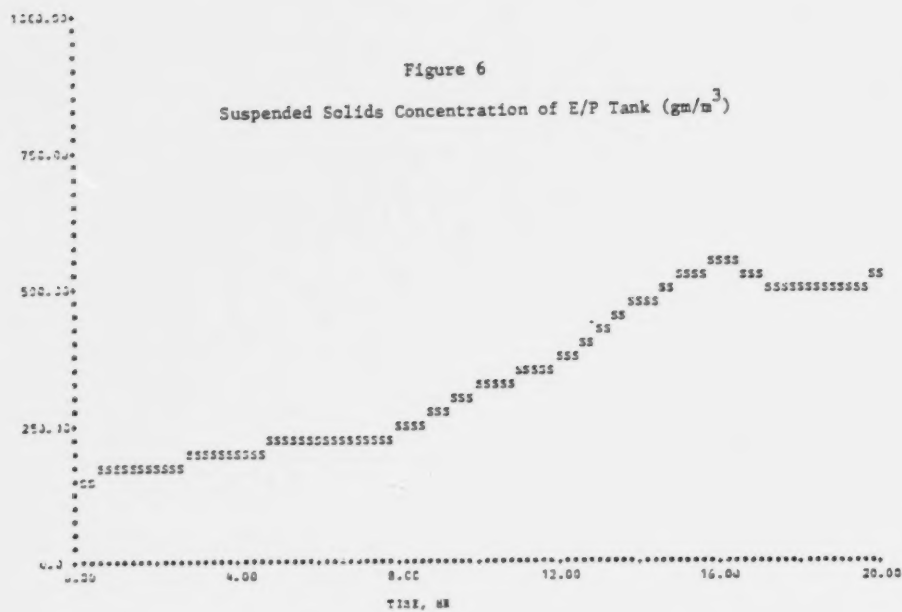


Figure 5. Expanded Ozonation and Hypochlorination Systems





AGRICULTURAL REUSE

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FOOD CROP IRRIGATION WITH RECLAIMED
MUNICIPAL WASTEWATER

Bahman Sheikh-ol-Eslami
Project Manager
Engineering-Science
600 Bancroft Way
Berkeley, California

William R. Kirkpatrick
Project Manager
Engineering-Science

Robert S. Jaques
Agency Engineer
Monterey Peninsula Water Pollution Control Agency
23845 Holman Highway, Suite 226
Carmel, California

INTRODUCTION

The Monterey Wastewater Reclamation Study for Agriculture (MWRSA) is an ambitious, long-term research and demonstration project created due to a unique combination of circumstances. MWRSA was created during the severe drought of 1976-1977 to show that food crops could be safely irrigated with reclaimed water.

Background

Setting. MWRSA is located between San Francisco and Los Angeles in one of the most beautiful coastal areas in California. The area is famous as a recreational and tourist resort area with breathtaking vistas and many historical landmarks as well as excellent golfing and water-oriented sports opportunities.

Agriculture. Food crops, such as lettuce, celery, artichokes, cauliflower, broccoli and tomatoes are grown almost exclusively in the north Monterey area. These crops have very high market values and are exported to markets throughout the United States. The crops are grown intensively and comprise a tremendous capital investment per hectare by the local farmers. What makes this investment level worthwhile is the region's excellent soil productivity and a year-round growing season.

Water Shortage. Increasing demands on the water supplies of North Monterey in recent years have produced significant shortages, especially affecting the agricultural sector. The irrigators have been forced to pump the ground water reservoir at greater rates and from deeper strata. This has resulted in declining water table levels which has in turn stimulated sea water intrusion into the coastal zone aquifers. Thus, over the last decades, agricultural water quality has gradually declined as salt water incursion has occurred, to the dismay of the agricultural industry.

Historical Chronology

The basinwide water quality management plan for Monterey County was prepared in 1973. Recognizing the water shortage and quality degradation problems, the plan recommended that all wastewater from the region

be consolidated for treatment at a regional plant and the effluent be reclaimed for agricultural irrigation. The facility planning study that followed corroborated that recommendation and, after evaluating 173 different plans, selected Alternative 166: an 83,000 m³/d (22 mgd) regional activated sludge plant near the mouth of the Salinas River, interceptors from the eight participating communities, effluent disposal through an ocean outfall in winter-time and reclamation of the effluent during the irrigation season.

At about the same time that the basin water quality plan and the facility plan were being developed, the Castroville Irrigation Project, a water importation and distribution scheme was also being developed to help relieve the water shortage problems. Thus, by the end of 1975 the local farmers were faced with the choice between reclaimed and imported water for their irrigation demand. In January 1976, a symposium was sponsored by the Farm Advisor's Office in Salinas with the participation of local farmers, several University of California professors, the Monterey Peninsula Water Pollution Control Agency (MPWPCA), state and county health agencies and other interested departments and groups. The purpose of this symposium was to provide authoritative answers to the many questions that the farming community had raised regarding potential problems with using reclaimed water. Predictably, the symposium did not completely allay the fears of the farmers; however, it did create a consensus that a long-term pilot demonstration project was needed before full-scale reclamation could be accepted by the farmers and the health authorities.

Thus was born the plan of study (1977) and the conceptual plan (1978) for the Monterey Wastewater Reclamation Study for Agriculture (MWRSA) which aims to demonstrate the feasibility of water reclamation in what is locally referred to as the "Artichoke Capital of the World."

OBJECTIVES

The most important objective of MWRSA is to generate demonstrable and unequivocal responses to concerns raised by the local farmers regarding water reclamation for food crop irrigation under the conditions at Castroville, California. These concerns include:

- Safety for the public and the farm workers vis-a-vis virus survival, bacteria and parasites, cadmium and other trace metals, aerosol transmission of pathogens, etc.
- Long-term soil impacts such as effects on permeability, structure, and salinity.
- Crop yield, quality and maturity.
- Consumer acceptance of food crops grown with reclaimed water.
- Economic feasibility of water reclamation, as related to necessary treatment level to satisfy health standards.

A second objective of the project is to evaluate treatment efficiency to meet the public health standards reliably. Another objective is to provide design criteria and operational experience for (1) the regional wastewater treatment plant and (2) the reclamation system for full-scale agricultural irrigation. Ultimately, however, the underlying

goal of the program is to demonstrate overall feasibility of wastewater reclamation for irrigation of vegetable crops that may be consumed raw.

OVERVIEW

The Monterey Wastewater Reclamation Study for Agriculture was in the planning stage for nearly two years, during which time the initial concepts received extensive review by a variety of concerned agencies, a large number of farmers, and members of the academic community. These planning efforts culminated in the publication of the project conceptual plan (Reference 1). It was established early on that to set up a pilot demonstration of water reclamation necessitated a pilot treatment plant that could duplicate the type of effluent expected from the planned future regional treatment plant. A farm, representative of the local agricultural soils, was needed and it had to be thoroughly characterized before the demonstration project was initiated. To answer the concerns and questions raised, it would be essential to set up experimental plots which could later be statistically compared in terms of tangible effects. It was decided that five years of data would be needed to provide adequate support for the conclusions of the study.

Furthermore, to demonstrate the feasibility of large-scale operations in a credible manner, it was also necessary to have large, field-size farms that were sprinkler irrigated with reclaimed water for the same five-year duration. At the end of the five-year period, the fields and plots used for the project would again be thoroughly characterized.

Ancillary Studies

In addition to the experimental and demonstration components of the project, a large number of ancillary studies are also being carried out at various times during the conduct of MWRSA. These ancillary studies include:

- evaluation of treatment processes employed;
- characterization of aerosols generated and transported;
- preliminary laboratory and environmental chamber studies of virus survival on plants, soils and in waters;
- monitoring agroclimatic conditions;
- health surveillance of project personnel;
- random sampling and analysis from neighboring fields;
- study of chlorine dioxide as a viral disinfectant;
- survey of consumer attitudes; and
- public education.

PILOT AWT PLANT

The existing 2,000 m³/d (0.4 mgd) Castroville Sanitation District treatment plant was selected for modification and upgrading to be used as the pilot AWT plant for use in MWRSA. This selection was made because (1) the plant was centrally located in the area of potential future full-scale reclamation for agriculture, and (2) the influent to the

plant was found to be similar to the projected composite influent to the regional treatment plant. With appropriate modifications, the secondary processes are being made representative of the planned regional treatment plant.

Process Selection

The California Administrative Code Title 22 specifies treatment processes that are legally required before reclaimed water can be applied to irrigate unprocessed food crops, such as lettuce. The law currently reads: "Reclaimed water used for the spray irrigation of food crops shall be at all times an adequately disinfected oxidized, coagulated, clarified, filtered wastewater." (Reference 2.) In a recent virus study conducted in Pomona, California (Reference 3), a far less expensive process train wherein the oxidized effluent is simply filtered using relatively low doses of coagulant as filter aid compared favorably with the required process train.

The difference between these two processes, in a large regional treatment plant, would be tremendous: if the State Department of Health should insist on the lengthier process, hereinafter referred to as "Title-22 effluent," it may become economically impossible to use the reclaimed water. On the other hand, if it can be shown that the filtered effluent stream is equivalent to Title-22 effluent vis-a-vis public health protection, the reclaimed water thus produced may be competitive in cost with imported water and other alternative supplies. This is the rationale for selection of the two processes in the design of the pilot AWT plant.

Design Considerations and Criteria

Process flexibility was provided for in the design stage to permit variation of the largest numbers of parameters for thorough process evaluation. Basic processes were designed to function in a steady-state mode. However, a hydraulic range of 4:1 and a chemical addition range exceeding 20:1 was deemed prudent for the design. These ranges would allow not only flexibility in varying the hydraulic loading rates, but also in changing chemical dose for process optimization and cost reduction. These flexibilities also allow temporal changes in the process operating parameters such that the various units may be operated at different hydraulic and organic loading rates for performance evaluation. These flexibilities will be utilized to help select the design criteria for the regional plant [83,000 m³/d (22 mgd)] should reclamation prove feasible and acceptable.

Filtered Effluent Flow Stream. The filtered effluent flow stream starts with secondary effluent from a complete mix activated sludge treatment plant, with alum and polymer in-line addition prior to dual-media gravity filtration. Following filtration, the flow stream moves through a plug-flow, long-detention, chlorine-contact tank. The purpose of the chemical coagulant/filter aid is to strengthen the floc so that in-depth filtration would be promoted. This would assure longer filter runs and resistance to shock loading.

The dual media filters used for the filtered effluent flow stream are in two separate filtration cells. This is : (1) to provide capability for cleaning and maintaining without shutting down the filtration process; and (2) to maintain the prescribed 3.4 l/m².s (5 gpm/ft²), surface loading during maintenance, low-flow periods, etc. This loading rate is one of the many grant conditions imposed by the California

State Water Resources Control Board which administers federal and state funds used for this project. Additional flexibility is available to bypass the filters altogether and test other filter systems and/or media as needed. Following filtration, the flow proceeds to a mixing basin and thence it is split into two separate compartments of the chlorine contact tank. This tank provides 90 minutes' contact time and has a length-to-width ratio exceeding 40.

At the outlet of the chlorine contact tank, the flow is controlled by weirs, conveying the flow into a three-compartment, over-and-under sulfur dioxide dechlorination basin.

Title-22 Effluent Flow Stream. As in the first flow stream, the Title-22 flow stream starts with secondary effluent from the Castroville activated sludge treatment plant. As secondary effluent enters the flocculator-clarifier, alum and polymer are added and are rapidly mixed in a static turbulent mixer prior to entering the slow-mix zone of the flocculator-clarifier, the reactor zone. Following a 20-minute detention time in this zone, flow enters the clarifier zone. Clarified effluent moves vertically upward and out into a 600-mm (24-inch) diameter columnar dual-media filter, which is completely manually operated.

Effluent from the filter then goes through a flash mixer and a serpentine 400-mm (16-inch) diameter pipeline which provides a 90-minute chlorine contact time. Sulfur dioxide is injected into the effluent from the chlorine contact tank before storage and distribution. Design criteria utilized in developing the plans and specifications for the pilot AWT plant are presented in Table 1.

THE EXPERIMENTAL COMPONENT

The experimental component of MWRSA will provide the quantitative answers to the primary objective of the project. Therefore, the experimental design follows agricultural research experimentation patterns (see, for example, Reference 4.) Seventy-two replicated trial plots will be set up with different combinations of irrigation water, fertilizer application and crop type. The treatments will be replicated four times. The principal variable of the experimental component is water type, as the relative effects of the reclaimed water are the prime objective of the study. The three treatments of this variable will be: (1) Title-22 effluent, (2) filtered effluent and (3) well water, for a control.

Fertilizer rate is another variable, applied at zero, full and partial rates as compared to rates commonly applied by farmers in the area. These treatments are designed to separate the fertilizer contributions of reclaimed water from any other crop yield effects that may be present. Finally, another variable is the crop type. A perennial crop, i.e. artichokes, will be grown in half of the plots on a permanent basis. In the other half, a series of other vegetable crops will be grown in a succession. The crops planned are lettuce, celery, broccoli and cauliflower: vegetables that are eaten raw.

Experimental Design

A "random-block" design is believed to be the most suitable experimental design under the constraints of the experimental material and populations involved. The random-block design is expected to provide the most statistically reliable data for assessment of the level of significance of relatively small differences expected in results emerging

from the various treatments. Some 2,000 samples of plant materials, soils and waters will be collected during each of the five years. These samples will be subjected variously to some of the 42 virological, bacteriological, chemical and physical analyses chosen for this project. Lists of sampling frequencies and analyses are shown in Tables 2 and 3.

THE DEMONSTRATION COMPONENT

Because of the nature of agricultural experimentation techniques, plots designed for the experimental component will be too small to demonstrate field-scale feasibility of irrigating with reclaimed water. Therefore, two actual production fields of the same crops, enumerated above, will be utilized in the demonstration component of MWRSA. These two fields will be four hectares (10 acres) each. In one field, artichokes will be grown perennially, whereas in the other, the same succession of lettuce, celery, broccoli and cauliflower will be grown simultaneously with those grown in the experimental component plots.

These larger fields are expected to be of particular interest to local farmers who want to know how reclaimed water can be used on their farms. The agricultural management practices will duplicate, exactly, those used by the area farmers. This is critical to the success of the demonstration objectives, because the farmers are generally reluctant to change their long-practiced operations to accommodate reclaimed water. Thus, irrigation and other management methods such as supplemental fertilization, pest control, equipment, implements, etc. will be identical with those used by the neighboring farmers.

ANCILLARY STUDIES

In addition to the experimental and demonstration components, MWRSA also includes several ancillary studies which are individually important as well as supportive of the essential components of the program. These studies were appended to MWRSA as questions were raised by the various participants during the conceptual planning stages.

Treatment Process Evaluation

The pilot treatment plant is designed with a great degree of process flexibility and sampling capability to enable process evaluation and adjustment. This will be of great value in the eventual design of the regional water reclamation system.

Characterization of Aerosols

In order to establish an adequate buffer zone around the demonstration fields, a series of tests have been performed at Castroville to determine the extent of airborne travel of aerosols generated during sprinkler irrigation. Automatic optical scanning devices were employed to enumerate particles in various size ranges from one to 100 μ m. Anderson samplers are now being used with a chemical tracer (lithium chloride) to determine bacterial content and to confirm travel distance of aerosols.

Preliminary Laboratory and Environmental Chamber Studies of Virus Survival

The need for controlled studies of virus deactivation arose when it became evident that a standardized, widely accepted procedure did not exist for field viral sampling. The climatic conditions of

Castroville are duplicated in an environmental chamber where plant and soil samples are dosed with known concentrations of bacteriophage and vaccine grade poliovirus. Recovery of virus at various elapsed times is proving very instructive for field sampling technique development. This topic is discussed in greater length further on.

Agroclimatic Conditions

A meteorological station was set up near the MWRSA field site to monitor air temperature, humidity, sunlight, evaporation and wind speed and direction. The data being produced will help establish irrigation schedules and other cultural practices and in addition will help interpret data from the experimental plots.

Health Surveillance

It is planned that routine medical examinations for all project personnel who will come into contact with the reclaimed water during the course of MWRSA will be provided. The Monterey County Department of Environmental Health Services will provide the examinations. While no problems are anticipated, any indication of potential disorder will be immediately reported, followed up and treated.

Random Sampling in Neighboring Fields

In order to control the experimental check plots, it will be necessary to sample the distant neighbors' fields at random times and locations. This information will strengthen knowledge of the prevailing and possible changing background conditions against which treated fields are being compared.

Chlorine Dioxide Disinfection

Preliminary studies have indicated that chlorine dioxide is by far more effective in virus deactivation than is chlorination. This important potentiality will receive a critical test during the first two years of MWRSA. During the non-irrigation season, the two separate chambers of the chlorine contact tank will simultaneously receive chlorine and chlorine dioxide respectively. The relative effectiveness of the two disinfection agents will be assessed by sampling at various elapsed contact times (0, 5, 15, 30, 45, 60, 75 and 90 minutes) along the contact chamber and analyzing for virus and bacteria. Virus seeding will supplement this study to establish the relative deactivation capability of the two chemicals under conservative conditions.

Both disinfection streams will also be tested for the possible formation of volatile organic compounds and trihalomethanes (THM).

Attitude Survey

Assurance of consumer acceptance of vegetables grown with reclaimed wastewater must be reasonably established before overall project feasibility can be determined. A preliminary search of published literature did not reveal any previous studies of consumer attitude toward food crops grown with reclaimed wastewater. Therefore, a study of consumer attitudes must be performed so that the marketability of such crops may be judged.

Grower attitude and assessments also are important elements of the proposed market study. The insights and experience of growers are

used in the design of consumer surveys and will assist in the evaluation of results.

The key element in the study of consumer attitudes toward purchase and consumption of produce grown with reclaimed wastewater will be a survey research program. Consumers of produce will be surveyed to determine their attitudes, and the results used to assess possible effects on demand of the products. The survey will be conducted no earlier than the fourth year of MW RSA.

The survey will be national in scope and will be directed toward households that purchase produce in a raw, unprocessed state for home consumption. The survey will concentrate on regions of the nation where a significant portion of produce sold originates in the Monterey area. A national survey is essential because it has been shown that vegetable consumption patterns vary between demographic subgroups, size and type of community, and regions of the nation. The regions of the country where the survey will be concentrated are Los Angeles, Chicago and New York.

A telephone survey will be used because costs are less than an in-person household survey, yet person-to-person contact is retained. Mailed questionnaires will not be used because of sampling problems and lower response rates. Approximately 2,000 telephone interviews will be necessary for results to be accurate within ± 10 percent in 95 out of 100 equivalently selected samples.

Public Education

A mass media educational program will be planned at the end of the public attitude survey in order to publicize the results of the study. Presumably, having amply demonstrated the safety of using reclaimed water for food crop irrigation, it may be possible to convince the consumers that the produce will be wholesome for human consumption.

INSTITUTIONAL INVOLVEMENTS

California's drought of 1976-1977 focused nationwide attention upon water reclamation projects. Several state regulatory agencies established special working divisions and set aside special funds to study methods of reducing demands on the state's dwindling potable water supplies. As a result of these circumstances, MW RSA received an intensive level of interest from a wide range of both public and private groups. This interest reached its peak during the conceptual planning phase of the project. To sustain the interest and participation of these various groups, while at the same time avoiding the costly delays frequently associated with the involvement of a large number of agencies, an organizational framework had to be developed to keep the project moving ahead successfully.

Institutional Objectives

The involvement of various regulatory agencies and private sector representatives was necessary to ensure that four major objectives of the project were accomplished. These objectives are:

- (1) To conduct the project in such a manner that its results would have widespread credibility and acceptance among the scientific and technical communities.

- (2) To conduct the project such that if the feasibility of full scale reclamation were demonstrated, farmers would be satisfied that they could successfully use reclaimed wastewater without fear of adverse public reaction in a highly competitive agricultural marketplace.
- (3) To promote local support and interest in the project so that local citizens (whose user charges would be helping to cover the costs of full-scale reclamation) would be aware of the local implications of reclamation and could provide constructive suggestions to guide the conduct of MWRSA.
- (4) To ensure that the requirements of the various grant funding agencies were being satisfied so that these revenues would continue throughout the life of the project, thereby keeping local costs to a reasonable level.

Formation of the Task Force

To determine which groups and agencies should be directly involved with the project so that each of these objectives would be achieved, inquiries were sent at an early stage to a number of local, state and federal agencies. The scope of the proposed project was briefly outlined and an indication of each agency's interest and ability to actively participate was solicited. Many of the responses pointed out the potential interest of other agencies or groups. Within a reasonably short time, a fairly comprehensive listing of interested groups was developed.

It was concluded early that close involvement by the state and county health departments and the State Office of Water Recycling would be essential to accomplish the first objective, i.e., widespread acceptance and credibility. In California, as in most other states, the state and county health departments must give their approval before any wastewater reclamation project may be put into operation. They establish health safety criteria on either a general or case-by-case basis for such projects, and require that comprehensive analytical data be provided to ensure that public health will not be endangered as a result of the use of reclaimed wastewater.

To accomplish the second objective, representatives of the local farming community, the County Agricultural Commissioner's Office and University of California agricultural scientists were asked to participate in the project. The farmers whose land would receive reclaimed wastewater for crop irrigation (if MWRSA demonstrated feasibility) asked that large numbers of agricultural groups not be involved initially. They were concerned that their marketing image could be adversely affected if it became widely known by the lay community that they were participating in the project. The participation of representatives of the County Farm Advisor's Office and the University of California agricultural experts strengthened the farmers' acceptance of MWRSA.

In California, the State Water Resources Control Board (SWRCB) administers state grant funding for all wastewater projects including reclamation projects. The Environmental Protection Agency grants the federal funds which are administered on its behalf by the SWRCB. Hence, close involvement of these two agencies was essential to the accomplishment of the fourth objective. A pleasant surprise came when it was learned that another state agency, the Department of Water Resources, was keenly interested in the project, and had special funds of its own to enable it to perform certain field work on MWRSA at no cost to the MPWPCA.

Managing the Project

Following initial institutional discussions, it was deemed necessary and useful to form an advisory Task Force, comprised of representatives of all the involved groups and agencies. Meetings were held regularly and frequently during much of the project's conceptual planning phase. The bulk of the technical work was performed by the MPWPCA's consultant, Engineering-Science, which prepared working papers and task reports for review and comment at each Task Force meeting. Each working paper discussed one or more key concepts to be addressed in the conduct of the project, and made specific recommendations. The content and recommendations of the working papers were frequently debated at length during Task Force meetings. Minutes of the meetings were kept, and after all questions and comments had been addressed, formal votes were taken which approved each of the documents. The working papers were ultimately edited and assembled into the "Conceptual Plan" (Reference 1). This document contains the detailed description of each phase of MWRSA.

The Task Force approach to involving the key agencies and groups has thus far proven to be a very satisfactory method of developing and implementing MWRSA at a satisfactory rate of progress. The Task Force meetings provide an open forum for the exchange of constructive comments. The continuing approval of each step in the project is assured by taking formal approval action on each issue or policy matter. That no significant procedural problems or delays have thus far occurred is testimony to the value of keeping all affected parties involved throughout the duration of the project.

PRELIMINARY ACTIVITIES AND FINDINGS

The planning phase of MWRSA was followed by a series of preliminary preparations, such as: site selection and acquisition; environmental assessment; case history development; literature search; right-of-way acquisition; topographic survey and initial site characterization. A detailed, monthly water quality characterization of the wastewater treatment plants in the region and irrigation water production wells has also continued to provide baseline data concerning the quality of the alternative water resources. Two especially significant preliminary activities, virus studies and aerosol monitorings, are described in greater detail below, with preliminary findings and indications that will have a significant impact on the detailed procedures to be used in MWRSA.

Virus Environmental Chamber Studies

Field survival of animal viruses on the produce is a crucial concern of the farmers, public health officials and of consumers in general. A survey of the literature reveals that at the present time no standardized methods exist for sampling virus from plant and soil materials. The need for development of new methods prompted the environmental chamber studies, initially and presently conducted by Professor Robert Cooper at the University of California, Berkeley, under a subcontract with Engineering-Science. The objectives of these preliminary studies are:

- (1) To perfect a method for the isolation of animal viruses from artichokes, lettuce and celery;
- (2) To set up, standardize and run the environmental chamber;

- (3) To evaluate a filter system for the recovery of viruses from large volumes of water; and
- (4) To determine bacteriophage and poliovirus survival on artichokes and other vegetables.

Initial findings of this preliminary virus study are highly encouraging. The first three objectives were successfully achieved, allowing determinations of virus and bacteriophage survival on vegetable matter under a variety of conditions. Quantitative data on "die-off" of viruses on artichokes contaminated with known concentrations of poliovirus and bacteriophage have already been obtained and are being confirmed with additional experiments. The initial tests produced a rapid die-off rate within the first few days, followed by a gradually decreasing rate: die-off, therefore, appears to be exponential in nature. Further observations will be performed before firm conclusions are drawn concerning survival of viruses on vegetation.

Aerosol Baseline Monitoring

Transport of pathogens away from the site of reclamation projects using sprinkler irrigation is an important issue. Work in this field is rather primitive and much needs to be learned before a safe minimum buffer zone width can be defined. Therefore, the Task Force guiding the conduct of MWRSA directed a thorough evaluation of the aerosol problem, as applicable to the Castroville conditions.

An optical scanning device was used to enumerate simultaneously aerosols in five particle size ranges over pre-set time periods. Nearly all aerosols generated were found to be smaller than 20 micrometers (μm) in diameter, with the majority in the 0.5 to 10 μm size range. A general, sharp decrease in the number of particles was observed with distance from the sprinkler lines in all but the smallest particle size ranges. The number of particles in nearly all size ranges remained fairly uniform within the range of normal human breathing height and began to decline at heights above four metres (13 feet).

Background aerosol counts in the Castroville area are generally so high, due to humidity and fog conditions prevalent on most days, that a definitive delineation between natural aerosols and sprinkler-generated aerosol is difficult. Future tests will include tracer studies of aerosol travel as well as the bacteriological content of the aerosols, with the use of Anderson Samplers. These data will provide a baseline for later comparison with aerosols generated when reclaimed water is used in the same fields.

With existing information available on aerosol generation in sprinkler irrigation in Castroville; with assumed pathogenic organisms occurrence probability in filtered secondary effluent; and with the physiology of the human tracheobronchial region, it was preliminarily estimated that the chances for inhaling or ingesting a viable pathogen during an eight-hour exposure period at a 15 m (50 ft) distance from the sprinkler line are one in 86,000.

REFERENCES

- (1) "Conceptual Plan, Monterey Agricultural Irrigation Demonstration Program," Engineering-Science, prepared for the Monterey Peninsula Water Pollution Control Agency (March 1978).
- (2) "Administrative Code Title 22, Division 3, Environmental Health," State of California (1975).
- (3) "Pomona Virus Study," Sanitation District of Los Angeles County, for California State Water Resources Control Board and the United States Environmental Protection Agency (February 1977).
- (4) Little, Thomas M. and F. Jackson Hills, "Agricultural Experimentation Design and Analysis," John Wiley and Sons, Santa Barbara (1978).

TABLE 1

DESIGN CRITERIA FOR PILOT AWT PLANT FOR THE MONTEREY WASTEWATER RECLAMATION STUDY FOR AGRICULTURE (MWRSA)

Item	Dimension	
	Metric	English
<u>Design Flows/Loading (Modified Secondary Plant)</u>		
Average dry weather flow	1,500 m ³ /d	0.4 mgd
Average design BOD ₅	400 mg/l	
Average TSS	250 mg/l	
<u>Design Flows (AWT Processes)</u>		
Average daily flow	946 m ³ /d	0.25 mgd
Title 22 flow stream	57 m ³ /d	0.015 mgd
Filtered effluent flow stream	889 m ³ /d	0.235 mgd
<u>Title 22 Treatment Facilities</u>		
<u>Chemical Coagulation</u>		
Flocculation		
Nominal detention time, minutes	20	
Tank volume	0.75 m ³	200 gal
Clarification		
Nominal detention time, hours	1.9	
Surface loading rate	32.6 $\frac{\text{m}^3}{\text{T}}$	800 gpd/ft ²
<u>Filtration</u>		
Number of filters	1	
Type of filter	gravity, dual media	
Sand		
Depth	300 mm	12 in
Effective size	0.45 - 0.55 mm	
Anthracite		
Depth	430 mm	17 in
Effective size	1.0 - 1.1 mm	
Filtration area	0.29 m ²	3.1 ft ²
Hydraulic detention time, minutes	90	
Length:width ratio	≥40	

TABLE 1 (Continued)

DESIGN CRITERIA FOR PILOT AWT PLANT
FOR THE MONTEREY WASTEWATER RECLAMATION STUDY
FOR AGRICULTURE (MWRSA)

Item	Dimension	
	Metric	English
Hydraulic loading rate @ ADWF	3.4 $\frac{1}{m^2 \cdot s}$	5.0 gpm/ft ²
Water backwash rate	13.6 $\frac{1}{m^2 \cdot s}$	20 gpm/ft ²
Supplementary backwash type	air scour preceding water backwash	
<u>Chlorination</u>		
Number of chambers	2	
Total tank volume	56.8 m ³	15,000 gal
Hydraulic detention time, minutes	90	
Length:width ratio	>40	
<u>Dechlorination</u>		
Chemical used	sulfur dioxide	
<u>Polymer Feed System</u>		
Maximum dosage to filter	5 mg/l	
Chemical used	liquid polymer	
<u>Dechlorination</u>		
Chemical	sulfur dioxide	
<u>Alum Feed System</u>		
Maximum dosages	200 mg/l	
Chemical used	alum	
<u>Polymer Feed System</u>		
Maximum dosage to flocculator & filter	5 mg/l	
Chemical used	liquid polymer	
<u>Filtered Effluent Treatment Facilities</u>		
<u>(New)</u>		
<u>Filtration</u>		
Number of filters	1	
Type of filter	gravity, dual media	
Filter media		
Sand--depth	300 mm	12 in
effective size	0.32 - 0.55 mm	
Anthracite--depth	600 mm	24 in
effective size	1.0 - 1.1 mm	
Filtration area	3.3 m ²	36 ft ²

TABLE 2
SAMPLING REQUIREMENTS FOR THE MONTEREY WASTEWATER RECLAMATION STUDY FOR AGRICULTURE
(MWSA)

Sample Requirements	Sample Materials									
	Irrigation Water (1)	Tail Water (2)	Ground Water (3)	Surface Soil (4)	Depth Soil (5)	Edible Plant Tissues (6)	Residual Plant Tissues (7)	Neighboring Plant Tissues (8)	Aerosols (9)	
(a) Time of sampling event	at irrigation	at irrigation	at cutback	after irrigation	after harvest	at harvest	at harvest	at harvest	at harvest	at irrigation
(b) Number of different sample types	3	1	1	1	4	1	1	5	3	
(c) Number of discrete samples of each type	1	45	3	45	45	45	45	1	1	
(d) Sampling events per year	20 to 30	20 to 30	1	20 to 30	2	2 or 3	2 or 3	2 or 3	20 to 30	
(e) Total number of samples per year ^a	60 to 90	b	3	900 to 1,350	360	90 or 135	90 or 135	10 or 15	c	
(f) Number of subsamples of each sample	4	3	3	3	4	3	3	3	1	

a Obtained by multiplying rows (b) x (c) x (d).
b Tailwater runoff will be deliberately minimized and it is expected that only about 10 samples/year will be required.
c Current plans are to sample aerosols only during the first year and when weather conditions are appropriate. A total of 20 to 30 samples is expected.
Note: Irrigation and tailwaters, soils, and edible and residual plant tissues will be sampled as shown from the statistical plots.

TABLE 3
TIMING OF ANALYSES TO BE PERFORMED ON THE VARIOUS SAMPLES COLLECTED EACH YEAR

Parameters	Irrigation Waters (1)	Tail Water (2)	Ground Water (3)	Surface Soil (4)	Sample Material				Residual Plant Tissues (7)	Neighboring Plant Tissues (8)	Aerosols (9)
					Depth Soil (5)	Edible Plant Tissues (6)					
Virus/Bacteriophage	I *	I	A	PI	A	H		H	H	H	-
Total Coliform	I	I	A	PI	-	H		H	H	H	I
Fecal Coliform	I	I	A	PI	-	H		H	H	H	I
Shigella/Salmonella	I	I	A	PI	-	H		H	H	H	I
Parasites	I	I	A	PI	-	H		H	H	H	I
pH	I	I	A	PH	-	-		-	-	-	-
Hardness	I	I	A	-	-	-		-	-	-	-
Sulfate	I	-	A	PH	A	-		-	-	-	-
Carbonate	I	-	A	PH	A	-		-	-	-	-
Bicarbonate	I	-	A	PH	A	-		-	-	-	-
Chloride	I	I	A	PH	A	-		-	-	-	-
Calcium	I	I	A	PH	A	-		-	-	-	-
Magnesium	I	I	A	PH	A	-		-	-	-	-
Sodium (adj SAR)	I	I	A	PH	A	-		-	-	-	-
Potassium	I	I	A	PH	A	H		H	-	-	-
Nitrate-N	I	I	A	PH	A	-		-	-	-	-
Ammonium-N	I	I	A	PH	A	-		-	-	-	-
Phosphorus (P ₂ O ₅)	I	-	-	PH	A	-		-	-	-	-

TABLE 3 (Continued)
TIMING OF ANALYSES TO BE PERFORMED ON THE VARIOUS SAMPLES COLLECTED EACH YEAR

Parameters	Sample Material						Residual Plant Tissues (7)	Neighboring Plant Tissues (8)	Aerosols (9)
	Irriga- tion Waters (1)	Tail Water (2)	Ground Water (3)	Surface Soil (4)	Depth Soil (5)	Edible Plant Tissues (6)			
Total Dissolved Solids	I ^{tr}	I	A	PH	A	-	-	-	-
Electrical Conductivity	I	I	A	PH	A	-	-	-	-
Boron	I	-	-	PH	A	H	-	-	-
Iron	A ^a	-	-	A ^a	A	H	-	-	-
Zinc	A ^a	-	-	A ^a	A	H	H	H	-
Copper	A ^a	-	-	A ^a	A	H	-	-	-
Cadmium	A ^a	-	A	A ^a	A	H	H	H	-
Nickel	A ^a	-	A	A ^a	A	H	-	-	-
Chromium	A ^a	-	A	A ^a	A	H	-	-	-
Mercury	A ^a	-	A	A ^a	A	H	-	-	-
Lead	A ^a	-	A	A ^a	A	H	-	-	-
Cobalt	A ^a	-	A	A ^a	A	H	-	-	-
Stable Organic Compounds	-	-	A	PH	A	-	-	-	-
Total Kjeldahl Nitrogen	-	I	A	-	-	H	-	-	-
Total Suspended Solids	-	I	-	-	-	-	-	-	-
Biochemical Oxygen Demand	I	I	-	PH	-	-	-	-	-
Cation Exchange Capacity	-	-	-	PH	A	-	-	-	-

TABLE 3 (Continued)
TIMING OF ANALYSES TO BE PERFORMED ON THE VARIOUS SAMPLES COLLECTED EACH YEAR

Parameters	Irrigation Waters (1)	Sample Material							Aerosols (9)
		Tail Water (2)	Ground Water (3)	Surface Soil (4)	Depth Soil (5)	Edible Plant Tissues (6)	Residual Plant Tissues (7)	Neighboring Plant Tissues (8)	
Organic Matter Content	-	-	-	PH	A	-	-	-	-
Clay Mineral Type	-	-	-	PH	-	-	-	-	-
Texture/PSD	-	-	-	PH	A	-	-	-	-
Permeability	-	-	-	PH	A	-	-	-	-
No. of Particles of Aerosol	-	-	-	-	-	-	-	-	I
Aerosol Particle Size Distribution	-	-	-	-	-	-	-	-	I

* Analyses to be performed on composite samples, collected throughout the irrigation season.

* Legend for time of sample collection:

I = time of each irrigation

A = annual, at the end of the irrigation season

PI = after each irrigation

H = at time of harvest

PH = after cut-back for artichokes

REUSE OF SEWAGE EFFLUENT IN URBANIZING IRRIGATED VALLEYS --

ARIZONA'S SALT RIVER VALLEY

Herman Bouwer
Director

U.S. Water Conservation Laboratory, USDA SEA-AR
4331 East Broadway, Phoenix, Arizona 85040

INTRODUCTION

The greater Phoenix area is water-deficient. There is considerable overdraft on the ground water, resulting in water-table declines that have been as high as 10 ft (3 m) per year in some parts of the Salt River Valley. Ground water furnishes about one-third of all the water used in the 250,000-acre (100,000-ha) area served by the Salt River Project. The rest or about 800,000 acre-ft (1 billion m^3) per year, is supplied on a renewable basis by the Salt and Verde Rivers. Six storage dams (four on the Salt and two on the Verde) store river water in times of high flow for use in times of low flow. Sewage production in the Salt River Valley is about 125 mgd (0.47 million m^3 /day) or 140,000 acre-ft/yr (173 million m^3 /yr) and increases each year by 6 mgd (23,000 m^3 /day) due to population growth. Sewage effluent thus is becoming a significant water resource that must be used.

Most of the sewage from Phoenix and surrounding cities is treated at the 23rd and 91st Avenue plants, which handle about 40 and 85 mgd (0.15 and 0.32 million m^3 /day), respectively. Both plants have primary and secondary (activated sludge) treatment without chlorination, and discharge their effluent into the normally dry Salt River bed where it flows west toward Buckeye. The ionic composition of the effluent is shown in Table 1.

The main effluent user presently is the Buckeye Irrigation Company, which has a contract for 30,000 acre-ft (37 million m^3) of effluent per year and diverts the effluent from the Salt River bed at Bullard, about 6 mi (10 km) downstream from the 91st Avenue Sewage Treatment Plant. Since not much effluent flows beyond the Buckeye diversion, loss of effluent due to seepage and evaporation by phreatophytes (mostly salt cedars) in the Salt and Gila Rivers must be considerable. This loss has been estimated at 30 per cent. Other effluent users are the Arizona Game and Fish Department, which is entitled to 7,300 acre-ft/yr (9 million m^3 /yr) to maintain a green belt in the riverbed, and the Flushing Meadows Project, which uses about 600 acre-ft/yr (0.74 million m^3 /yr) for experimental ground water recharge. The major future user of sewage effluent will be the Palo Verde nuclear power plant that is being constructed about 45 mi (72 km) west of Phoenix. As the population continues to increase and more effluent will become available, use of effluent for irrigation may be expanded and it may also be used for recreational lakes.

NUCLEAR POWER PLANT

The Palo Verde nuclear power plant has a contractual arrangement for an ultimate use of 140,000 acre-ft (173 million m^3) of effluent per year for cooling. The effluent will be transported directly from the 91st Avenue Treatment Plant to the site with an underground pipeline. The

first three reactor units, each with a capacity of 1270 MW, are scheduled to go on-line in 1982, 1984, and 1986. The units would use about 100,000 acre-ft (123 million m³) of cooling water per year. Ultimately the plant will have five reactor units. The effluent will be treated on-site, using trickling filters to nitrify NH₄ and to reduce alkalinity, two-stage lime softening to raise the pH and precipitate Ca, Mg, and SiO₂, followed by addition of CO₂ to reduce the pH and precipitate excess Ca as CaCO₃, and gravity filtration. The treated effluent then flows into an 80-acre (32 ha) reservoir with a capacity of 1,950 acre-ft (2.4 million m³) -- sufficient to keep three units operating at peak production for 8 days. The cooling system will operate at about 15 cycles of concentration, after which the brine will be disposed in evaporation ponds.

The nuclear power project pays the City of Phoenix \$1 per acre-ft (1,233 m³) of effluent prior to using it for cooling to preserve its option to the wastewater. This amount will be raised to \$2 per acre-ft after the official permit is issued. Once the power plant uses the water for cooling, it will pay a price that is equal to 40 per cent of the cost of municipal and industrial water from the Central Arizona Project, but not less than \$20 and no more than \$30 per acre-ft (1,233 m³).

IRRIGATION

Use of sewage effluent for crop irrigation is subject to state regulations, which specify the type treatment or quality criteria that an effluent must meet before it can be used to irrigate certain crops. Arizona standards,* for example, allow the use of undisinfected secondary effluent for irrigation of fiber or forage crops not intended for human consumption, and for orchard crops if the effluent does not contact the fruit (overhead sprinklers cannot be used). Since this is the effluent that the Buckeye Irrigation Company diverts from the Salt River bed, the company is restricted as to the types of crops that can be irrigated with the effluent. If the secondary effluent is chlorinated or otherwise disinfected to produce a total coliform concentration of less than 5,000/100 ml and a fecal coliform concentration of less than 1,000/100 ml, it can be used to irrigate crops for human consumption, provided the product is cooked or otherwise processed to destroy pathogenic organisms. Such effluent can also be used to irrigate orchard crops if the effluent does contact the fruit, and golf courses and cemeteries. If the secondary effluent has received additional treatment (for example, tertiary treatment, lagooning, or ground water recharge) that reduces both the BOD and suspended solids content to less than 10 mg/l and the fecal coliform concentration to less than 200/100 ml, the effluent can be used for unrestricted irrigation (including sprinkler irrigation of fruit and vegetables eaten raw by humans). Such effluent can also be used to irrigate parks, lawns, school grounds, private yards, and sport fields, or to provide a substantial portion of water in lakes used for primary-contact recreation. California standards† are more stringent. For unrestricted irrigation, for example, they require that the effluent be adequately disinfected (7-day median

*Arizona Department of Health Services Rules and Regulations, Chapter 20, Title 9, Article 4.

†California Administrative Code, Chapter 4, Title 22, Division 4, Environmental Health.

coliform concentration not in excess of 2.2/100 ml and 3-day maximum coliform concentration not in excess of 23/100 ml), well-oxidized (organic matter stabilized), coagulated (colloidal and finely divided suspended matter removed), clarified (clarification of oxidized, coagulated effluent by further settling), and filtered (clarified wastewater which has passed through soils or filter media).

The above standards for crop irrigation with sewage effluent are based on public health aspects. However, agronomic factors must also be considered. For example, the nitrogen concentration of the effluent may be too high for general crop irrigation, because it could produce undesirable effects such as too much vegetative growth and not enough fruit or seed production, lodging, delayed maturity and harvestability (cotton), reduced flavor and poor texture (fruit and vegetable crops), reduced sugar content (beets or cane), or reduced starch content of potatoes.¹ According to Ayers,² nitrogen in irrigation water causes no problems if its concentration is below 5 mg/l, increasing problems if it is between 5 and 30 mg/l, and severe problems if it exceeds 30 mg/l. Since secondary sewage effluent normally contains 20 to 40 mg/l nitrogen, agronomic problems can be expected if certain crops are irrigated with such effluent. Crops like hay and forage crops can, of course, tolerate and benefit from large amounts of nitrogen. Other quality parameters that must be considered in using sewage effluent for irrigation are the total salt content (no problems below 500 mg/l, increasing problems between 500 and 2,000 mg/l, and severe problems above 2,000 mg/l), the sodium adsorption ratio SAR (no problems below 6, increasing problems between 6 and 9, and severe problems above 9), and the concentration of phytotoxic ions like sodium, fluoride, boron and heavy metals.²

In addition to public health and agronomic aspects, aesthetic factors such as appearance, odor, and treatment technology must also be considered, particularly if the irrigated areas are in or near regions with a high population density. Complete removal of pathogens would be preferable over merely reducing the fecal-coliform density to a certain value. This is especially important when sewage water is to be used for lakes with primary-contact recreation. Recreational use would also require a drastic reduction in the phosphorus content of the water -- perhaps not to as low a value as the threshold concentration for algal development (0.05 mg/l PO_4-P), but certainly as close to that value as practicable.

The additional quality improvement necessary to make sewage treatment plant effluent suitable for unrestricted irrigation and primary-contact recreation is typically obtained by filtering the effluent through natural soils and aquifers, as achieved by ground water recharge with infiltration basins. For the Salt River Valley, this was demonstrated with the Flushing Meadows Project, which is an experimental rapid-infiltration system installed in 1967 in the Salt River bed about 1.5 miles (2.4 km) downstream from the 91st Avenue Sewage Treatment Plant.³⁻⁵ The project consists of six parallel infiltration basins 20 by 700 ft (6.1 by 213 m) each and spaced 20 ft (6.1 m) apart. The soil consists of 3 ft (0.9 m) of fine, loam sand underlain by sand and gravel strata to a depth of 240 ft (76 m) where a clay layer begins. The water table is at a depth of about 10 ft (3 m). In 10 years of research, this project has shown that:³⁻⁷

1. The largest hydraulic capacity of the system was obtained with flooding periods of 2 to 3 weeks alternated with drying periods of 10 to 20 days. At this schedule and using a water depth of 1 ft (0.3 m) in

the basins, the hydraulic loading rate was 400 ft/yr (122 m/yr). Vegetation or gravel layers in the basins did not offer particular advantages and the best bottom condition was bare soil. Volunteer vegetation did not seem to have any undesirable effects, so weed control was not necessary. Sludge-like accumulations on the basin bottoms were periodically removed. After 10 years of operating the project and a total infiltration of 2473 ft (754 m), there were no signs of reductions in hydraulic loading or in hydraulic conductivity of the underlying aquifer.

2. Nitrogen removal from the effluent water as it seeped through the ground to become renovated water was about 30 per cent at maximum hydraulic loading, but 65 per cent if the loading rate was reduced to about 200 ft (60 m) per year by using 9-day flooding and 12-day drying cycles and by reducing the water depth in the basins to 0.5 ft (15 cm). The form and concentration of nitrogen in the renovated water sampled from the aquifer below the basins were slow to respond to a reduction in hydraulic loading. In the tenth year of operation (1977), the renovated water contained 2.8 mg/l $\text{NH}_4\text{-N}$, 6.25 mg/l $\text{NO}_3\text{-N}$, and 0.58 mg/l organic N, for a total-N content of 9.6 mg/l. This is 65 per cent less than the total N-content of the secondary sewage effluent, which averaged 27.4 mg/l (mostly as $\text{NH}_4\text{-N}$) in that year.

3. Phosphate removal increased with increasing distance of underground movement of the sewage water. Removal after 30 ft (9 m) of downward movement to and in the underlying aquifer was about 40 per cent at high hydraulic loading and 80 per cent at reduced hydraulic loading. Additional lateral movement of 100 ft (30 m) through the aquifer increased the removal to 94 per cent in 1977 (i.e., a concentration of 0.51 mg/l of $\text{PO}_4\text{-P}$ in the renovated water versus 7.9 mg/l in the effluent). After 10 years of operation and a total infiltration of 2473 ft (754 m), there were no signs of a decrease in phosphate removal.

4. Fluoride removal paralleled phosphate removal. Fluoride concentrations in 1977 were 2.08 mg/l for the effluent, 1.66 mg/l for renovated water sampled from wells between the basins, and 0.95 mg/l for renovated water sampled from a well 100 ft (30 m) away from the basins.

5. Boron was not removed and was present at concentrations of 0.5 to 0.7 mg/l (0.59 mg/l in 1977) in both effluent and renovated water.

6. Concentrations of zinc, copper, cadmium, lead, and mercury were below maximum limits for drinking and permanent irrigation, except for lead, whose concentration of 0.066 mg/l in the renovated water exceeded the 0.05 mg/l limit for drinking water but was well below the 5 mg/l limit for permanent irrigation.

7. BOD_5 -values were reduced from 10 to 20 mg/l for the effluent to essentially zero for the renovated water. Corresponding COD -ranges were 30 to 60 and 10 to 20 mg/l, respectively. Not all organic carbon was removed from the effluent water as it seeped through the soil and aquifer. The TOC contents of the renovated water were 5.2 mg/l for the wells between the basins and 3.3 mg/l for the wells 100 ft (30 m) away from the basins. The TOC content of the secondary sewage effluent was 19.2 mg/l. Trace organics in renovated wastewater presently are the main concern in potable use of such water.

8. The secondary sewage effluent contained 21 virus units (PFU's) per liter, but viruses could not be detected in renovated water sampled below the basins. Fecal coliforms were present at concentrations of up to several hundred per 100 ml in the renovated water sampled below the basins, but could not be detected in renovated water sampled 300 ft (90 m) away from the basins. Hence, a lateral movement of about 330 ft (100 m) through the aquifer appears adequate to produce renovated water free from fecal coliforms. Laboratory studies showed that distilled water (rain!) could remobilize previously adsorbed viruses in the soil during the first 24 hours of a drying period, but that addition of CaCl_2 prevented most of this desorption. This finding is important for managing rapid-infiltration systems in humid climates.

Rapid-infiltration systems thus can produce renovated sewage water that meets the requirements for unrestricted irrigation from the standpoint of public health, agronomic effects, and aesthetics. This is advantageous when it is desirable to upgrade the quality of the sewage effluent so that it can be used to grow vegetables and other crops that are more profitable than crops that can be irrigated with secondary sewage effluent as such. This is the rationale for the 23rd Avenue recharge project, which consists of four 10-acre (4 ha) rapid-infiltration basins to renovate secondary effluent from the 23rd Avenue Treatment Plant by ground water recharge (Figure 1). The water table is at a depth of 50 to 100 ft (15 to 30 m), and renovated water will be pumped from three wells on the center dike. The well in the center is already installed. It is perforated from 100 to 180 ft (30 to 55 m) depth and delivers about 2500 gpm (157 l/s). The other two wells will be located symmetrically on the center dike (Figure 1). The three wells will pump renovated water into a common underground discharge pipe which eventually will transport the water to a canal of the Roosevelt Irrigation District. The infiltration basins were constructed in 1975 by installing three parallel dikes within an existing 40-acre (16-ha) oxidation pond to divide it into four equally-sized basins. Observation wells were installed on the north and south sides of the project (Figure 1) to monitor ground water levels so as to avoid or minimize movement of renovated water into the aquifer outside the system by managing infiltration and pumping rates.^{1,2}

The main problem with the 23rd Avenue rapid-infiltration project has been an increase in the algae concentration of the secondary effluent as it passed through the 80-acre (32 ha) oxidation pond before entering the infiltration basins. This was particularly serious in the summer, when the suspended solids content of the secondary effluent was around 15 mg/l when it entered the 80-acre (32 ha) pond, but between 50 and 100 mg/l when it left the pond and entered the infiltration basins. The algae were mostly of the single-cell type and did not settle out at the inlet end of the basins. Instead, they formed a "filter cake" on the entire bottom of the infiltration basins. The hydraulic impedance of this algae cake was very high, so that the resulting hydraulic loading rates in the basins were only about 72 ft (22 m) per year, which is much less than the 200 to 400 ft (60 to 120 m) per year obtained at the Flushing Meadows Project.³ The basins of the Flushing Meadows Project were long and narrow and had a certain outflow rate to keep the detention time sufficiently short to minimize growth of suspended algae.

To remedy the algae problem at the 23rd Avenue project, a by-pass channel has just been constructed in the 80-acre (32-ha) pond (Figure 1) to bring secondary effluent from the treatment plant directly into the infiltration basins. This should increase the hydraulic loading rate

and make it possible to predict the true hydraulic capacity of the system. Once this is known, the Roosevelt Irrigation District can design the pipeline to carry the renovated water to its canal about 0.5 mile (800 m) north of the project. If, for example, the hydraulic loading rate will be 350 ft (106.7 m) per year, the project could produce 14,000 acre-ft (17.3 million m³) of renovated water per year, or 12.5 mgd or 8,680 gpm (548 l/s).

The Roosevelt Irrigation District has a contractual arrangement to obtain from the City of Phoenix 20,000 acre-ft (24.7 million m³) per year of renovated sewage effluent (enough to irrigate about 5,000 acres or 2,000 ha). The water already has been certified by the State Health Department as being suitable for unrestricted irrigation. The District will pay the City of Phoenix 75 per cent of the average price that the Salt River Project charges its customers for irrigation water on class A agricultural land. This charge was \$6.00 per acre-ft (1233 m³) in 1978, so that the cost of the renovated water then would have been \$4.50 per acre-ft (1233 m³). The Roosevelt Irrigation District will also pay the cost of pumping the renovated water from the wells, which is estimated to be about \$5 per acre-ft (1233 m³) for power.

RECREATION

Recreational use of renovated sewage water will primarily be in conjunction with the Rio Salado Project. This project was originally proposed in the mid-1960's by the College of Architecture of Arizona State University to revive the normally dry Salt River bed with a chain of lakes and other amenities, "as a Phoenix bird rising from its ashes." The proposal remained dormant for a few years, however, because suitable water was not available. When it became apparent that sewage water could be rather inexpensively renovated by rapid-infiltration systems and that the resulting renovated water would be suitable for primary-contact recreation, the plan was revived. A Rio Salado Steering Committee was formed under the auspices of the Valley Forward Association, and funds were collected for conceptual planning by a Los Angeles consulting firm with support from local firms. A report on the Phase II Planning Study was released in July 1974. The Rio Salado Phase III Study was completed in the summer of 1977 by the Research and Service Foundation of the College of Architecture, Arizona State University.

The Phase III Study covers a 16-mile (26 km) stretch of Salt River bed from Country Club Drive in Mesa to 27th Avenue in Phoenix. It encompasses 8,442 acres (3,377 ha) of land area (mostly riverbed) with 2,032 acres (813 ha) in lakes. Other amenities include ground water recharge areas (possibly combined with wildlife sanctuaries), a sky ride connecting the Arizona State University campus with the Phoenix airport and downtown Phoenix, a parkway, marinas, equestrian centers, scout camp, tennis centers, auto course, golf courses, and other recreational and athletic facilities. The lakes would also function as flood channel when water is spilled from the dams on the Salt and Verde Rivers. To achieve this, the dams for the Rio Salado lakes should be collapsible or have sufficient spillway or gate capacity to accommodate "floods." Upstream flood control would also be necessary to protect the Rio Salado Project against disastrous floods of the 50- to 100-year recurrence interval type.

The Rio Salado Project offers tremendous potential for beautification of the Salt River bed and for providing aquatic and other recreation in the "back yard" of the greater Phoenix area. It would also unite the

north and south parts of the valley now separated by the Salt River. A lot of time, money, and dedicated effort will be required before the project is a reality.

URBAN PLANNING FOR WATER REUSE

To facilitate wastewater renovation by ground water recharge and subsequent use of the renovated water for various purposes, urban development in irrigated valleys should be concentrated on the desert, foothills, or other unirrigated land surrounding the valley (Figure 2). The sewage treatment plants could then be located on alluvial fans, near ephemeral streams, or other places on the valley perimeter suitable for rapid-infiltration systems. The renovated water from these rapid-infiltration systems would move through the aquifer to the lower parts of the valley where it could be pumped for irrigation and other uses (Figure 3). The aquifer thus would serve as a medium for storing and transmitting renovated water.

An urbanization scheme as in Figure 2 will require considerable advanced planning. Legal, social, and economic obstacles must be resolved. Without such planning, however, urbanization of irrigated valleys tends to take place on the irrigated land around existing towns in the valley, using up valuable agricultural land and making the older portions of the towns increasingly vulnerable to flooding due to storm runoff from the newer and higher developments around them. Sewage treatment plants in such valleys tend to be large and located at the lower end of the valley near the river. This would not favor reuse of sewage plant effluent in the valley itself, because it would require costly pump-back systems. For additional details regarding urbanization of irrigated valleys according to the models in Figures 2 and 3, reference is made to Bower.¹⁰

SUMMARY

Sewage effluent is an important water resource in arid areas such as the Salt River Valley in Arizona. Most of the effluent from the greater Phoenix area will be used for a nuclear power plant. Secondary effluent will be piped to the plant, treated on-site, used for cooling, and then put in evaporation ponds. Irrigation use can be expected to increase. Ground water recharge with rapid-infiltration basins and pumping the resulting renovated sewage water from aquifers is an effective and economical way to upgrade the effluent quality so that it can be used for unrestricted irrigation. It will then also be suitable for recreational lakes, which are planned as part of the Rio Salado Project. Concentrating urban development around, instead of in, irrigated valleys facilitates ground water recharge with sewage effluent and reuse of the water within the valley.

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TABLE 1
Concentrations of Ions in Secondary Sewage Effluent⁴

Ion	Concentration mg/l
Na ⁺	200
Ca ²⁺	82
Mg ²⁺	36
NH ₄ ⁺	40
K ⁺	8
HCO ₃ ⁻	381
CL ⁻	213
SO ₄ ²⁻	107
PO ₄ ³⁻	30
NO ₃ ⁻	2
CO ₃ ²⁻	0
TDS	1000
pH (units)	8

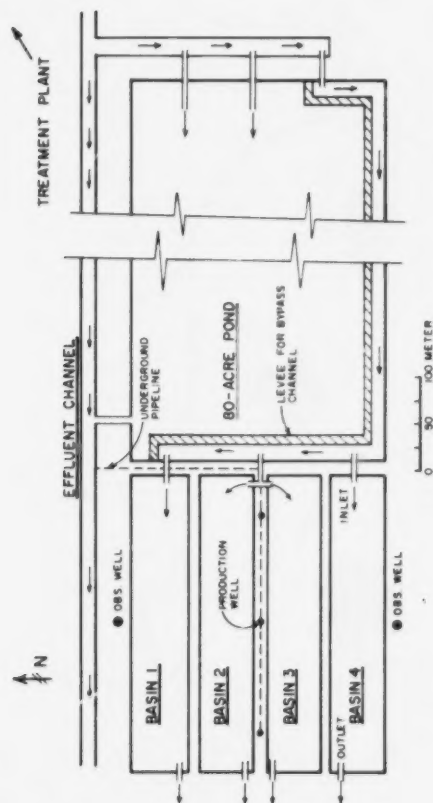


Fig. 1. Schematic of 23rd Avenue Rapid-Infiltration Basins Showing By-Pass Channel in 80-Acre Pond, Four Infiltration Basins, Observation Wells, and Production Wells for Pumping Renovated Water.

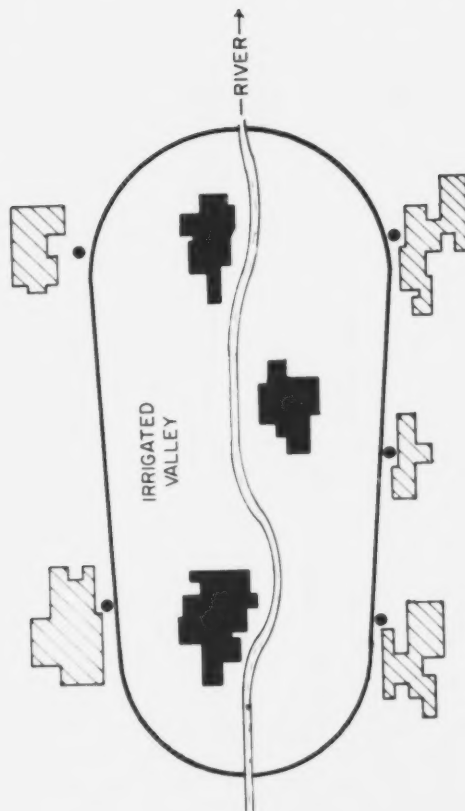


Fig. 2. Schematic of Irrigated Valley with River, Old Towns (Black) in Valley, and New Towns (Hatched) with Local Sewage Treatment Plants (Black Dots) Around Irrigated Valley.

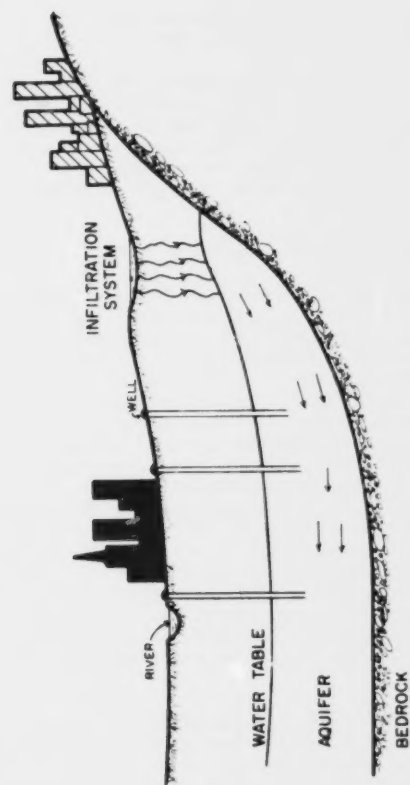


Fig. 3. Cross-Section Normal to River of Valley in Figure 2, Showing Old Town (Black) and New Town (Hatched) with Rapid-Infiltration System and Movement of Renovated Water Through Aquifer to Wells in Lower Part of Valley.

PLANT PATHOGEN CONTAMINATION
IN REUSED IRRIGATION WASTE WATER

James R. Steadman
Associate Professor of Plant Pathology
University of Nebraska-Lincoln

Rick W. Bay
Engineer, Nebraska Department
of Environmental Control

Mark J. Hammer
Professor of Civil Engineering
University of Nebraska-Lincoln

The critical need for irrigation water emphasized by the recent droughts, the need for increased irrigation efficiency, and governmental regulations have increased the demand for reuse. An inevitable consequence of water reused after passing through agricultural crop production fields with diseased plants or infested soil is that water may become contaminated with, and thus, disseminate plant disease-producing organisms.

DISTRIBUTION OF PHYTOPATHOGENS IN IRRIGATION WATER

Only a few studies have shown that plant pathogenic fungi are in reused municipal waters (1,8). The occurrence of fungal, nematode, and bacterial pathogens in irrigation water, however, has been reported throughout irrigated regions of North America. In fact, irrigation return flow has been shown to be of primary importance in the dissemination of many phytopathogens. Fungal pathogens have been found in Georgia and North Carolina irrigation reuse ponds (7,12,14). *Phytophthora* spp. in irrigation systems has been reported from British Columbia (13) and the Pacific Northwest (11) to southern California (10) and Arizona (17). Other phytopathogenic fungi such as *Verticillium albo-atrum*, *Sclerotinia sclerotiorum*, *Rhizoctonia solani*, and *Sclerotium* sp. have been reported in irrigation waste water (2,14,15). Plant parasitic nematodes were found in large numbers in the irrigation canals of south central Washington (4). Further studies indicated that the primary source of those nematodes was irrigation return flow (5). A study in western Nebraska in the North Platte Project found a bacterial pathogen of bean, *Xanthomonas phaseoli*, in addition to various saprophytic and parasitic nematodes in all parts of the irrigation system (15).

Recent studies by the senior author have confirmed contamination of the inadvertent irrigation reuse water in the North Platte Project. Additional studies have sampled designed reuse systems in west central, central, and eastern Nebraska. From more than 50 waste water or waste water reuse samples collected, nearly 60 per cent of corn fields showing symptoms of Goss's bacterial wilt (*Corynebacterium nebraskense*) had waste water contaminated with *C. nebraskense* as detected in a pathogenicity bioassay. In two instances, detection of the organism in the runoff water preceded observation of disease symptoms in the field.

The capacity of some phytopathogens to survive in water for relatively long periods of time assures that they can contribute to dissemination. Shokes and McCarter (14) have shown that fungal spores or other fungal resting structures or fungi in host tissue can survive the winter season in surface irrigation ponds (85-185 days). Even though phytopathogenic bacteria have no special protective structures and are poor saprophytic competitors, many can survive for more than 30 days in irrigation water (Figure 1). Thus, phytopathogen survival and the design of reuse systems assures that pathogens can move both long and short distances in agricultural waste water.

CROP LOSS CONSIDERATIONS

Phytopathogens have many methods of dissemination in addition to irrigation water. These include: rain, wind, on or in seed or propagative material, man, and animals. Thus, it is difficult to determine experimentally the economic loss resulting from the reuse of contaminated agricultural waste water. A study in the Columbia Basin demonstrated that field plots irrigated with canal water became heavily infested with a variety of plant parasitic nematodes while field plots irrigated with well water did not become infested (6). In Oregon, irrigation of onions by surface drainage water with bacterial contamination resulted in increased crop losses in field and storage when compared to onions irrigated with well water (9). In many instances, the potential of the disease-producing organisms in irrigation water has not been measured but remains a serious threat to the irrigated crops. With designed reuse systems, the irrigator often takes drainage water off a field which had, for instance, corn in it the previous year and then applies this water to a new corn field. This practice could assure that this year's corn will be exposed to last year's corn diseases.

DETECTION OF PHYTOPATHOGENS

As mentioned above, a number of phytopathogens have been identified in specific geographic areas. However, at the present time these pathogens must be identified on a field-by-field and pathogen-by-pathogen basis. Attempts to find a simple indicator for waterborne plant pathogens analogous to the coliform test for potable water have not been successful. This is not surprising in that phytopathogens are not linked to a common origin as are the enteric bacteria and viruses. Fungi and nematodes can be identified by morphological spore forms and by adult or cyst structure, but this is neither fast nor simple. Phytopathogenic bacteria are more difficult to identify in that morphological criteria and selective media are often not available. Many phytopathogens can be disseminated in infested plant tissue, which further complicates identification. In deriving the data presented in this paper, the authors isolated antibiotic resistant strains of *X. phaseoli* and *C. nebraskense* to enable experiments to be conducted in natural irrigation water.

REDUCTION OF PHYTOPATHOGENS IN IRRIGATION WATER

Of the various potential methods for minimizing plant pathogen contamination of reuse water, chlorination may be the simplest and least expensive. Chlorination has been used to reduce the development of postharvest fruit diseases, and the methodology has been reviewed (3).

However, very little information has been reported for utilizing chlorine for disinfection of irrigation water. In 1965, bacterial stalk rot of corn was reduced on a North Carolina farm when pond water applied by sprinkler was treated with sodium hypochlorite providing a 1 mg/l residual (17).

The dearth of information on chlorinating agricultural water necessitated determining water quality effects on chlorination. Reuse water samples were assayed by standard procedures for temperature, pH, turbidity, total alkalinity, hardness, solids (suspended, volatile and total), ortho phosphate and nitrate (Table 1). Ammonia plus organic nitrogen and pH had the most significant effect on chlorine disinfection. The pH levels of reuse water (6.3 to 8.6) covered a critical range affecting the disinfection efficiency of chlorine. Organic plus ammonia nitrogen was found at levels up to 10 mg/l estimated by breakpoint chlorination, and interfered with chlorination by reacting rapidly with chlorine, forming chloramines of relatively weak disinfection capabilities.

Water samples were separated into three types by breakpoint chlorination analysis as shown in Figure 2. Type I water was easily disinfected by low levels of chlorine (5 mg/l or less). Type II water required careful control and monitoring of chlorination at 5 to 16 mg/l to provide a free residual for rapid disinfection. Type III water, where combined chlorine was formed even at high dosage levels, required a long contact time for disinfection; free chlorine being 700 times more effective in disinfection than the combined form. Most water samples were classified as Type I or Type II with only a few samples classified as Type III. Phytopathogenic bacteria react similarly to *Escherichia coli* in that low levels of free chlorine were adequate for disinfection (Figure 3). A free chlorine residual of 0.01 to 2.0 mg/l, depending on pH and contact time, or a combined chlorine residual of 5 to 10 mg/l provided 99 per cent kill. Spores of fungal pathogens, however, required 10-15 mg/l free chlorine for 5 minutes at pH 6.5 for a 99 per cent kill (Figure 4).

Pathogens located within host tissue are protected from chlorine. Thus, effective chlorination may depend on adapting other methods such as sedimentation or filtration to eliminate protected pathogens before chlorine treatment. Effective chlorination of reuse water can be obtained in the absence of protected pathogens by injecting directly into the return irrigation pipe.

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TABLE 1

Maximum, Minimum, and Mean Values of Water Quality Tests
Performed on Irrigation Reuse Water Samples

<u>Test</u>	<u>Maximum</u>	<u>Minimum</u>	<u>Mean</u>
pH	8.7	6.3	7.9
Turbidity NTU	500	15	101
Alkalinity			
as mg/l (CaCO ₃)	215	70	140
Hardness			
as mg/l (CaCO ₃)	300	60	149
Ortho-Phosphate			
as mg/l (PO ₄)	12.4	0.0	2.1
Nitrogen			
Nitrate = Nitrite			
as mg/l (N)	10.0	0.0	1.9
Temperature			
Degree Celsius	32.0	19.0	25.6
Total solids			
mg/l	2780	222	525
Suspended Solids			
mg/l	415	7	108
Volatile Suspended			
Solids mg/l	53	1	20
Estimated Nitrogen			
Organic + Ammonia			
as mg/l (N)	10	0.0	1.4

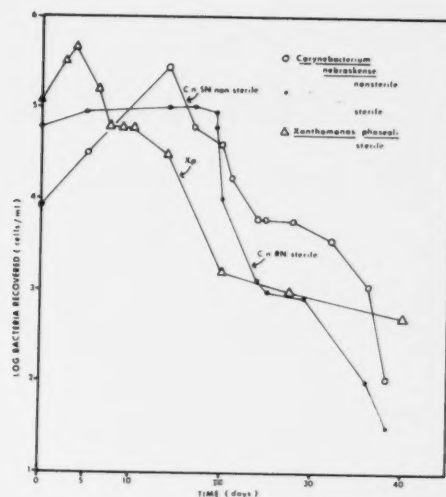


Figure 1. Survival of populations of *Xanthomonas phaseoli* and *Corynebacterium nebraskense* strains in sterile and nonsterile typical irrigation reuse water at from 0 to 40 days. Each point is an average of two experiments. S = streptomycin resistant, R = rifampicin resistant, N = nalidixic acid resistant.

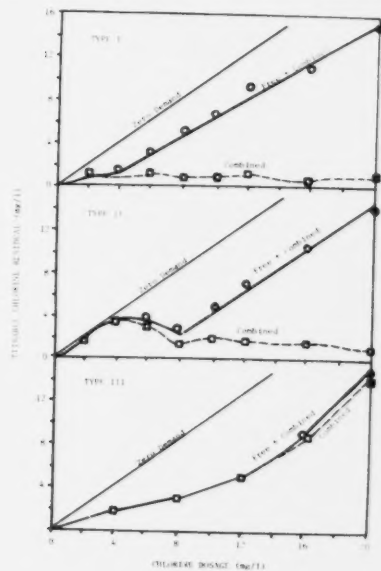


Figure 2. Type I, Type II, or Type III classification of water samples by breakpoint chlorination analysis.

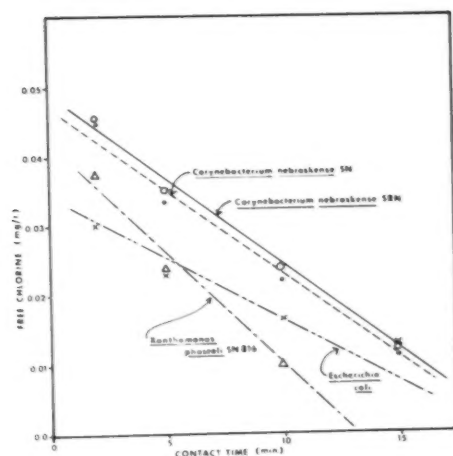


Figure 3. Free chlorine residual in irrigation water needed to kill 99 percent of the cells of *Xanthomonas phaseoli* strain SN816, *Corynebacterium nebraskense* strains SRN and SN and *Escherichia coli* after contact times from 2 to 15 min. Each point is an average of three experiments except for those for *E. coli* which represent 1 experiment. S = streptomycin resistant, R = rifampicin resistant, N = nalidixic acid resistant.

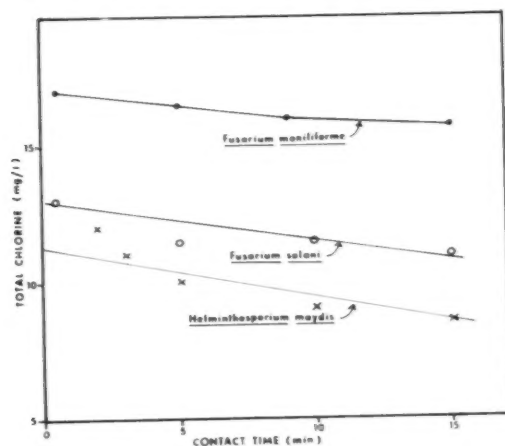


Figure 4. Chlorine residual needed to kill 99 percent of the spores of *Fusarium moniliforme* (macroconidia), *Fusarium solani* (macroconidia), and *Helminthosporium maydis* at contact times from 0.5 to 15 min. Each point is an average of 3-7 experiments. The pH was 7-8.

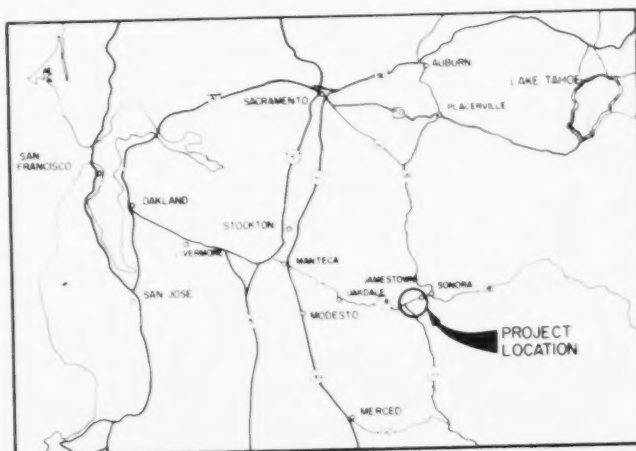
IRRIGATING PRIVATE LAND
WITH WASTEWATER EFFLUENT

James D. Farles
Senior Water Resources Engineer
CH2M HILL
Sacramento, California

Larry C. Amans
Water Resources Engineer
CH2M HILL
Sacramento, California

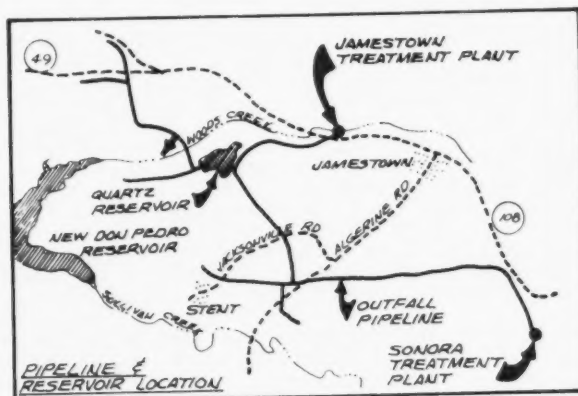
INTRODUCTION

In 1971 the California Regional Water Quality Control Board adopted requirements which prohibited sewage effluent discharge to tributaries above Don Pedro Reservoir in central California. Sewage treatment plants in Tuolumne County, California were subsequently ordered to eliminate discharges to Woods and Sullivan Creeks. The responsible agencies decided to plan a regional project and selected Tuolumne County Water District No. 2 (TCWD No. 2) to lead the effort. Planning, design, construction, and operation aspects of the effluent disposal system for the regional project are described in this paper.



PLANNING

A 1972 facility plan report called for effluent disposal on land in the seemingly worthless Red Hills, a barren federally-owned area 10 miles (16 km) southwest of Sonora. A state geological report later indicated that subsurface fractures made the hills unsuitable for effluent disposal. However, additional planning was not done until the regional collection and treatment facilities were under construction in 1974. At that time purchase of 1,000 acres (400 ha) of rangeland was proposed as the land disposal site. Following a series of heated public hearings, stacks of petitions, and a privately-funded advertising campaign against the proposal, TCWD No. 2 scratched the plan.



In 1975 CH2M HILL was hired to find an acceptable effluent disposal alternative. An initial report was prepared on available viable alternatives. These included direct release to a stream which discharged below Don Pedro Reservoir, winter storage with irrigation of TCWD No. 2 purchased land, and winter storage with delivery as irrigation water to individual ranchers. Even though the new proposals eliminated the concept of land disposal that had so inflamed county residents, they met with chilly public response. Many landowners admitted their need for irrigation water, but sewage effluent was still considered dangerous or repugnant. For nearly a year TCWD No. 2 Manager Les Delbon and CH2M HILL engineers met with small groups of opponents to the plan. The personal contact and honest dialogue gradually earned the trust and interest of detractors.

EPA and California clean water grant funds were used to prepare facility plan and environmental impact reports on the effluent disposal system. Based on results presented in those reports, TCWD No. 2 selected the irrigation water delivery to individual ranchers alternative for design and construction.

The selected reuse plan is unique in that it is operated as an irrigation system rather than a wastewater disposal system. Water is delivered to individual ranchers based on a pre-determined irrigation schedule, adjusted each year for cropping patterns, rainfall, etc. Wastewater flows are regulated by an 1,800 acre-foot (2.2 million m³) reservoir so that water deliveries can meet irrigation requirements. Deliveries are made through 19 automated turnouts along a 9-mile-long (14 km) outfall pipeline.

Each rancher has signed a separate contract with the District to take a specified quantity of water (dependent on irrigated acreage) each year during the irrigation season (April through October). The water is free but the rancher agrees to take the water for 20, 30, or 40 years. The contract is also binding on successive property owners. If a rancher does not take the water, the District may enter the land and apply the water as specified. To provide system flexibility each rancher has agreed to take all water made available up to 125 percent of the contracted amount although the District is obligated to make every effort to temporarily reduce the water delivered in any given year if there are cropping pattern changes.

Ultimately over 30 ranchers with a combined 1,300 acres (530 ha) will be using the effluent primarily for pasture irrigation. Acceptable uses of the secondary effluent, according to the California State Department of Health, include irrigation of pasture, fodder, fiber or seed crops plus stockwatering and esthetic uses where public contact is prohibited.

DESIGN

Design of the effluent disposal system began in the fall of 1976 and was completed in the fall of 1977. The project facilities include a 9 mile (14 km) effluent outfall pipeline, an 1,800 acre-foot (2.2 million m³) irrigation storage reservoir, a computer based telemetry system, and a 15 hp (11 kW) pumping plant.

Land application area and irrigation water storage requirements are based on projected effluent flows and local cropping patterns and soil characteristics. Effluent flows from the Sonora and Jamestown treatment plants of 1.0 mgd (3,800 kiloliters/day) in 1977 are projected to increase to 4.0 mgd (15,000 kiloliters/day) in 1995 with a 1995 peak 3-hour

design flow of 8.0 mgd (30,000 kiloliters/day). Shallow soils and the area's steep slopes limit the primary agricultural use to pasture grasses with a computed water requirement of 2.87 acre-feet/year per acre (3,540 m³/year), resulting in an ultimate land requirement of 1,300 acres (530 ha). An active storage volume of 1,500 acre-feet (1.9 million m³) is required to contain winter flows for subsequent summer release. Dead storage below the lowest outlet increases the total reservoir size to 1,800 acre-feet.

The major project facilities are shown above. Effluent flows by gravity from the Sonora Treatment Plant (elevation 1607) to Quartz Reservoir (max water surface elevation 1387). A 15 hp (11 kW) pump station at the Jamestown Treatment Plant (elevation 1334) lifts effluent to the reservoir. Irrigation deliveries are made from the pipeline below the reservoir as well as from the Sonora-to-reservoir pipeline. During the non-irrigation season the Sonora effluent flows directly to the reservoir.

The outfall pipeline is asbestos cement transmission pipe, Class 30, ranging from 6-inch to 24-inch diameter (150 to 610 mm). Nineteen on farm irrigation turnouts along the pipeline provide irrigation water to 33 ranchers. Turnouts consist of a valve and propeller meter in a locked access manhole followed by a 2-inch to 10-inch (50 to 250 mm) pipe riser with blind flange for subsequent connection by the rancher. Alternatively, an overflow standpipe arrangement was provided to some ranchers for direct discharge to on-farm lateral ditches.

A unique feature of the irrigation system is the automated operation of irrigation turnouts. Solenoid actuated, hydraulically operated globe valves on each pipe turnout are remotely operated by command from a control panel at the Sonora Treatment Plant. Commands are transmitted by FM radio signal and initiated on a keyboard at the treatment plant. Flow rates and totalized flows at each turnout can be displayed on a cathode ray tube screen or line printer upon command at the treatment plant control panel.

To maintain full-pipe flow and fairly constant pressure at irrigation turnouts, a 16-inch (410 mm) solenoid controlled throttling valve was installed in the pipeline just upstream of the reservoir. It maintains a constant liquid level at the Sonora Treatment Plant regardless of flow rate in the pipeline. If enough turnouts along the Sonora-to-reservoir pipeline are open to take the full plant discharge, the valve will close. When no turnout valves are open during winter months, the throttling valve will open to allow all water to discharge to the reservoir.

Quartz Reservoir provides in-line storage of winter effluent for subsequent downstream irrigation use. The 100-foot-high (30 m), 1,000-foot-long (300 m), Quartz Dam is of zoned embankment earthfill construction. An impermeable clay core constructed of onsite materials is flanked by upstream and downstream main shells of onsite gravelly clays.

A telemetry system was designed to remotely control and record farm turnout deliveries as well as to monitor system flow and liquid levels at important check points. The telemetry system consists of a central console station which controls and accepts data from 10 remote control stations via FM (173.3275 megahertz) radio communication links.

The central station at the Sonora Treatment Plant provides automatic polling of status data from remote units every 132 seconds and maintains a constantly-updated record of this data. An operator at the central station may, in addition, operate irrigation turnouts at each remote station with simple keyboard commands. Other keyboard commands provide displays of system status on a cathode ray tube (CRT) display or on a hardcopy line printer. All alarm conditions detected result in an audible warning tone and condition messages on both the CRT and the printer.

The system incorporates extensive safeguards to insure data and command integrity. All remote stations will continue to operate through a 4-hour main power outage. Communications are verified through error-checking codes incorporated into messages. Operator commands are verified by duplication before execution at a remote station.

The central station consists of a Neptune/UES Model 4100 microprocessor with a Beehive Model B100 CRT terminal and Centronics Model 703 printer all located in a desk enclosure with printer and CRT on the desk top. Remote telemetry stations consist of a Neptune/UES Model 4100 or 490 microprocessor linked to a GE MVP radio transceiver with 50 foot (15 m) wood pole antenna.

CONSTRUCTION

Construction began in the fall of 1977. The work was phased with separate contracts for: (1) the Sonora-to-reservoir pipeline, (2) the pipeline below the reservoir, (3) Quartz Reservoir and (4) the telemetry system. The work was divided into smaller parts to allow local contractors a better chance at landing a part of the \$3,980,000 project. Funding was provided 75 percent by the Federal government, 12-1/2 percent by the State of California, and 12-1/2 percent by the owner, TCWD No. 2.

The Sonora-to-reservoir pipeline construction was the first contract to begin in November 1977. The contractor encoun-

tered difficult trenching conditions since the terrain is mountainous and almost the entire pipeline route was through hard Sierra Nevada greenschist and phyllite formations within 1 to 2 feet of the ground surface. The contractor elected to blast almost the entire 5 mile (8 km) trench length followed by conventional trenching operations.



The pipeline section below the reservoir and from the Jamestown Treatment Plant to the reservoir was constructed at the same time as Quartz Reservoir starting in early 1978 and finishing in the fall of 1978. An unusual feature of the reservoir construction was the placement of the 26-acre (11 ha) asphalt membrane lining. The ground surface was first graded then compacted with four passes of a sheepsfoot roller and four passes of a rubber tire roller to ensure a uniform base for the liner, free of high points and other irregularities. The oxidized petroleum asphalt, conforming to ASTM D2521, was applied in three passes by truck tanker with sideboom spray bar (see above). Hand directed sprayers were used for touchup work.

The telemetry system is currently under construction and slated for completion in late spring of 1979.

OPERATION

During the 1979 irrigation season, fully automated delivery of treated wastewater will begin to private ranchers in the Sonora area. Initial irrigation deliveries were made during the summer of 1978 on an interim basis to 15 ranchers for application to 150 acres (61 ha) of pasture. These deliveries were made by manually opening turnout valves. At the same time, about 10 percent of the effluent was discharged to the reservoir area for use by the reservoir contractor in earthfill compaction operations.

Ultimately over 30 individual ranchers will be served. With the telemetry controlled system, operation personnel requirements are reduced. The operator merely depresses a key on the console for a given turnout, then enters the desired flow rate, from 100 gpm (63 l/s) to 1,300 gpm (82 l/s) and the remote station transmits actual flow data to the CRT for visual confirmation. The operator can also call up flow and liquid level data to be presented in tabular form on the lineprinter thus eliminating the need for manual reading and recording of this data.

Meetings between District personnel and ranchers will be held each February or March to determine the water delivery schedule. Based on effluent quantities in storage at that time, estimates of summer flow, and ranchers' acreages and crops, a schedule of releases will be formulated. Free, nutrient-rich water will be provided on a long-term basis solving both ranchers' water supply problems and the District's disposal problem.

IRRIGATING PRIVATE LAND WITH WASTEWATER EFFLUENT

CH2M HILL

SACRAMENTO, CALIFORNIA
JAMES D. EARLES LARRY C. ABAMS

OBJECTIVES

- DESIGN AND CONSTRUCT WASTEWATER EFFLUENT TREATMENT PLANT
- TREAT WASTEWATER EFFLUENT TO A LEVEL
- TREAT WASTEWATER EFFLUENT TO A LEVEL

DESIGN FEATURES

- TREAT WASTEWATER EFFLUENT TO A LEVEL
- TREAT WASTEWATER EFFLUENT TO A LEVEL
- TREAT WASTEWATER EFFLUENT TO A LEVEL

CONSTRUCTION FEATURES

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CONCLUSIONS

- TREAT WASTEWATER EFFLUENT TO A LEVEL
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VISION OF BALANCE
A CITY ECO SYSTEM

Marvin D. Thurber
Director of Operations
City of Westminster, Colorado

It has not been easy for America's cities. Population growth, urban sprawl, water and energy shortages, high taxes, skyrocketing prices - all have contributed to the urban crisis.

The City of Westminster continues to search for solutions, but there are no magic formulas.

Growth along the Front Range of Colorado has been phenomenal in the past decade. Development has been accompanied by the predictable problems - inadequate services, environmental degradation and increased public debt.

In addition to these constraints, the Front Range has to cope with a very limited supply of the most elemental of human needs - water. A burgeoning population, accompanied by increased per capita demand for water, poses a twin threat to the area's future - to its vitality and to the quality of life it can provide.

MANAGEMENT THE KEY

Meeting the problem head-on, the City of Westminster, Colorado, has committed itself to a program of Growth and Resource Management - a program which city officials believe will not only accommodate future growth, but at the same time improve the community's quality of life, while limiting public investment for water and wastewater to an acceptable level.

A suburban community situated northwest of Denver, Westminster has grown from a population of 19,500 in 1970 to approximately 47,000 in 1978. Its utility service area contains more than 65,000 persons. About 10,000 acres, or 50 per cent of the land within the city limits, are currently undeveloped.

From fewer than 200 housing starts per year in the early sixties, building permit applications exploded to a potential 3,500 single family units in 1977.

By the mid-seventies, it became apparent that city resources, both physical and social, could not continue to absorb the impact of uncontrolled growth. Major concerns were the shortage of raw water, new environmental wastewater problems and the ever-increasing cost to the citizen as the demand for public services grew. What will be the cost of a subdivision in terms of new demands on the capacity of the system (libraries, police, fire, street improvements, traffic lights, utility pump stations, reservoirs)? Will the development generate sufficient revenue through taxes and service charges to cover these costs? What impact to the environment will occur? What impact will growth have on the city's infrastructure, the physical system through which services are provided?

It was the task of the city administration and the city council to devise a plan which would maintain a balance between all elements of

the urban system - the manmade physical environment, the natural environment and the fiscal environment.

The result was the Westminster Growth and Resource Management Plan - a program which enables the city to base development and management decisions on a comprehensive investigation of the physical, natural and fiscal environments as they relate to one another.

A SIX-PRONGED PLAN

The plan is composed of six components - each inextricably linked to the other.

- 1) The establishment of realistic growth levels which take into consideration the physical, environmental and fiscal capacities of the city;
- 2) Evaluation and prioritization of developed and undeveloped land within the city limits for its impact upon the total city;
- 3) An exhaustive analysis of the fiscal impact of any new development;
- 4) Initiation of programs to combat air pollution;
- 5) Encouraging the conservation of the water supply via a number of specific plans; and
- 6) A successive water use scheme which will increase both the city's raw water supply and irrigation water for the farmer downstream.

First, the city's potential growth level for a fixed period of time in the future has been established by analyzing all available resources on the city's physical, environmental and fiscal capacities.

Zoning is modified for planned unit development. All undeveloped property in the city is mapped into two and one-half tracts, and each tract analyzed for its physical, environmental and fiscal characteristics. Each area of land, therefore, can be evaluated in terms of the impact of proposed plans for new development.

Second, a system of land use priorities was developed and computerized. Present use of developed land was analyzed and evaluated, revealing certain imbalances in existing land use. To overcome these imbalances, city officials developed a system of priorities as a basis on which all future development would take place. For example, land contiguous to urban development is given higher priority for development than land not contiguous to an urbanized area.

All undeveloped land in the city has been evaluated according to these criteria and is assigned one of five priorities for development. Whether land may be developed and how it may be developed is based on an analysis of all components of the urban system.

TAX POTENTIAL A FACTOR

A third component of the Growth and Resource Management Plan is fiscal planning. An exhaustive analysis of the city's fiscal capabilities shows how potential growth may be implemented and assists city offi-

cials in establishing priorities for and limitations on future growth. It is imperative that the city determine the fiscal impact on its citizens of all new development. "What is the bill going to be?" and "Who will pay the bill?" are two questions which must be answered when any future development is considered.

Air quality, the fourth component, has become a problem of some magnitude in the Denver Metropolitan area, and programs to improve air quality have a high priority. Westminster has taken the lead in Front Range Denver by initiating the following programs and projects:

- 1) A program to provide designated car pooling parking areas throughout the city;
- 2) An effort to reduce vehicular travel by promoting shopping within the city limits of Westminster;
- 3) A program to encourage employers to purchase car pooling vans for their employees;
- 4) A campaign to encourage the use of bicycles as a means of transportation to work and for local shopping;
- 5) An educational program for local school systems to encourage good habits which can reduce pollution;
- 6) A lobbying effort to improve R.T.D. bus routes;
- 7) Cooperation with state and federal legislators in their efforts to enact stronger measures to combat air pollution in the Denver area;
- 8) Installation of emission-testing equipment to test all city vehicles on a scheduled basis, as well as offering free testing to the public on a pre-announced and scheduled basis; and
- 9) A study of the feasibility of providing shuttle bus service to Westminster area shopping centers from various points in the city.

Water conservation, the fifth component of the Westminster Growth and Resource Management Plan, is one of the most critical and limiting factors in the community's growth. Because water in Colorado is a permanently scarce resource, it must be the goal of city government to reduce water consumption and to encourage water conservation by residents.

As growth levels for the future were being developed, it became evident to city administrators that new policies in regard to water sources and water usage must be developed.

CONSERVATION PROGRAMS AUTHORIZED

Westminster's first attempt at water management was aimed at reducing overall demand by cutting per capita consumption of water. To implement the fifth component, the city council authorized the following programs which have been implemented by various city departments:

- 1) Inexpensive water-saving devices for use in private homes have been made available to all present Westminster water customers. Payment for the devices is incorporated into the customer's utility bill.

2) All new developments are now required to install water-saving devices, including insulation of hot water piping, in all structures. Under the building permit allocation plan, developers may obtain "bonus units" of building permits by adopting water conservation measures, such as common areas, outside the superstructure. The city will also grant bonus units for partial "dryscaping" of lots, soil preparation efforts and the construction of water-saving lawn irrigation systems.

INVERTED RATES EFFECTIVE

3) To encourage conservation, Westminster has adopted an inverted water and sewer rate structure, whereby the price per thousand gallons of water increased with each thousand gallons delivered to a customer. In addition, the price increase is substantial for water when usage is above the historical average amount necessary for a single family home with an average size yard. Through this restructuring of the water rate, the price of water is reduced for small volume users, increases moderately for the medium user and rises considerably for the high volume users. The city also eliminated the minimum water bill and the flat rate, so users pay only for what they consume.

The success of Westminster's water conservation efforts is obvious when one notes that annual per capita consumption has decreased by 22,000 gallons - a 15.0 per cent reduction. Within this figure, the inverted rate structure accounts for a reduction of 10.0 per cent from the expected growth rate (as measured by surrounding communities), while water-saving devices have contributed an additional 5.0 per cent savings.

It is noteworthy that Westminster is the only major city in the Denver metropolitan area which has not had to resort to water rationing.

In this semi-arid state, the search for new water supplies is constant, and, of necessity, must be expanded to include both conventional and unusual sources. A result of the search by Westminster has been an innovative successive water use program, the first of its kind in Colorado and one of the first in the nation.

WATER EXCHANGE AND SUCCESSIVE USE PLAN

This unique program, the sixth and final component of the city's Growth and Resource Management Plan, allows Westminster to use water earmarked for agricultural irrigation in its domestic water system, later replacing that water downstream with effluent from its wastewater treatment plant.

Under the terms of an agreement signed in 1976, with the Farmers' High Line Canal and Reservoir Company, Westminster is allowed to divert into its Standley Lake reservoir farmer's water from the High Line Canal. The water is then treated for domestic use in the city's system.

An equal amount of effluent from the city's Big Dry Creek Wastewater Treatment Plant is returned via pipeline to the canal downstream for use as irrigation water by area farmers.

This program adds significantly to the city's water supply without adversely affecting agricultural production. It provides a method whereby urban and rural residents, working together, can utilize the available water supply in a form most suitable for their individual needs.

The successive use program has also produced a number of tangible benefits for the farmers downstream.

EFFLUENT BRINGS ADDED BENEFITS

Utilizing Westminster's effluent for irrigation of crops is not only cost-effective for the farmer suffering from a fertilizer shortage but is beneficial for his crops. Virtually all plant nutrients essential to growth can be found in desirable quantities in the effluent. It not only meets stringent health standards (through constant monitoring and control), but it has been found to be superior to the water presently in the canal with respect to bacterial quality and BOD demand.

Yet another advantage to the farmer is that he is provided with a regulated flow of irrigation water, especially beneficial during dry years when the stream can be regulated to provide a sufficient and dependable irrigation water supply from the beginning to the end of the growing season.

Westminster has also contracted with area farmers to apply stabilized sludge, the by-product of wastewater treatment, to their farmland as a soil conditioner and liquid fertilizer. Taking into consideration the current shortage of energy requirements for producing inorganic fertilizer, experts agree that sludge recycling for agricultural purposes offers one of the best opportunities to reduce agricultural energy demands.

Immediate expansion of Westminster's wastewater treatment plant from 1.0 mgd to 2.4 mgd was necessary to implement this program. Future expansion to 5.0 mgd is in process now. The immediate expansion, including construction of an effluent pump station and force main for land application, was aided by a 75 per cent federal grant, thus making this the first federally funded successive use project in Colorado.

WATER SUPPLY AUGMENTED

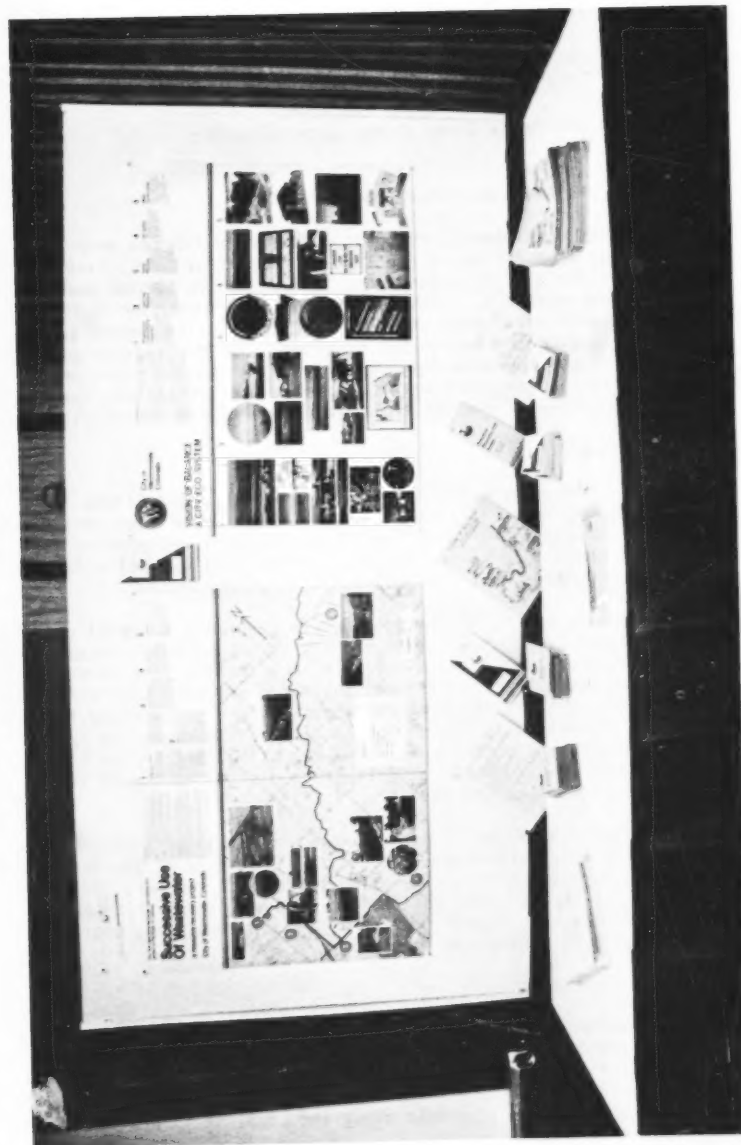
The successive use plan and a search for other new water sources have produced an additional 800 acre feet of water in 1978, enough to serve 4,400 persons in this rapidly growing city. Future expansion of the water exchange program is expected to yield an additional 5,000 acre feet of water by 1989.

Based on conservative estimates, Westminster's water system could then serve an additional 35,000 residents, a remarkable feat, given the lack of new water sources in the area.

To meet the needs of today and tomorrow, new directions must be taken and new solutions found. Westminster's unique Utility Task Force with a remarkable and committed City Council has recognized that the urban system, like the ecological system, is composed of interdependent factors which can be managed most effectively by maintaining the intricate balance that exists between them.

Only with such a comprehensive approach can the city continue to grow, yet preserve a high quality of life at an affordable price for the people within its boundaries.

Westminster is committed and is today meeting the challenge of tomorrow - with a Vision of Balance.



WATER REUSE FOR IRRIGATION
GILROY, CALIFORNIA

Lloyd C. Fowler
Chief Engineer
Santa Clara Valley Water District

Why was the reclamation system constructed?

The system was developed in response to the need for new water supplies in the Gilroy area following the 1976-77 drought and resultant severe overdraft of the limited groundwater table. As the second year of the drought became apparent, continued decline in groundwater levels was expected. Those areas with shallow wells and shallow depths to bedrock could be expected to run out of water. The reclamation project water supplements the groundwater supply by making two cubic hectometres per year of recycled water available for irrigation, leaving that much groundwater for use in homes, commerce and industry.

Treatment process:

The domestic sewage from the Cities of Morgan Hill and Gilroy are given standard primary treatment at the Gilroy Sewage Treatment Plant. Effluent from this plant is delivered to the water reclamation facility, which consists of a six hectare surface area oxidation lagoon, disinfection process, and a pipeline distribution system.

The oxidation lagoon is about three metres deep with a normal operating water depth of about 2.5 metres. Its rectangular six hectare area is divided in two by a redwood baffle wall along the long dimension--about 330 metres. Primary effluent is pumped into the southerly half of the lagoon, travels the length, passes through a submerged hole in the end of the wall into the northerly half, and returns the length of the lagoon to the outlet pump. The detention time in the lagoon is about 12 to 18 days. The oxidation process in the lagoon is assisted by eight floating aerators.

From the oxidation lagoon, the treated water is pumped through mechanical strainers and is chlorinated. The first 3000 metres of the distribution system pipeline acts as the chlorine contact chamber. At the end of this reach, the chlorine residual is measured to assure sterilization. If adequate chlorine residual is not maintained, the system is shut down by an automatic control.

Distribution System:

The distribution system consists of about 12,800 metres of 300mm diameter, cement mortar-lined and coated, steel pipe. There are 23 irrigation service turnouts on the pipeline ranging from 75 to 150 mm in size. Surge pressure relief and butterfly isolation valves are located at appropriate intervals along the pipeline to protect the distribution system.

Along the pipeline is a three-pair control cable. In the first 3,000 metres, there are two cables to provide for the chlorine residual monitoring unit feedback to the pumping station. The control

cable provides information on reservoir elevation, valve position, and flow rates.

The distribution system pump at the oxidation lagoon delivers 76 litres per second against a head of 80 metres. Provision is made for installation of a second pump when water use increases. Water is delivered directly from the pipeline through irrigation turnouts and into the pre-existing 120,000 cubic metre Debell Reservoir.

The reservoir provides the system with the flexibility to meet peak season irrigation demands. It is used to store water during off irrigation demands. It is used to store water during off irrigation periods and drawn upon during peak periods when the capacity of the pump at the oxidation lagoon would be exceeded. A water level sensor in the reservoir is used via the telemetry cable to operate the system within acceptable reservoir fluctuations.

WATER QUALITY REQUIREMENTS FOR WATER REUSE

Type of Use	Treatment Requirements ^a	Allowable Number of Coliform Bacteria per 100 ml.	
		Median ¹	Maximum ²
Spray irrigation of food crops	Oxidation ³ , coagulation, clarification, filtration, and disinfection	2.2	23
Surface irrigation of food crops	Oxidation and disinfection	2.2	---
Surface irrigation of orchards and vineyards ⁴	Primary treatment ⁵	---	---
Irrigation of fodder, fiber and seed crops	Primary treatment ⁵	---	---
Irrigation of pasture for milking animals	Oxidation and disinfection ⁷	23	---
Landscape Irrigation: Golf courses, cemeteries, freeway landscapes and landscapes in other areas of similar public access	Oxidation and disinfection ⁷	23	240 ⁶
Landscape Irrigation: Parks, playgrounds, schoolyards and other areas of similar public access	Oxidation, coagulation, clarification, filtration and disinfection	2.2	23
Nonrestricted recreational impoundment	Oxidation, coagulation, clarification, filtration and disinfection	2.2	23
Restricted recreational impoundment	Oxidation and disinfection	2.2	---
Landscape Impoundment	Oxidation and disinfection ⁷	23	---

1 The median value shall be determined from the bacteriological results of the last seven days for which analyses have been completed.

2 Shall not be exceeded in more than one sample in any 30-day period.

3 Oxidation is taken to mean biological treatment.

4 No fruit shall be harvested that has come in contact with the irrigation water or the ground.

5 Shall not be exceeded in any two consecutive samples.

6 Methods of treatment other than those listed and their reliability features may be accepted if the applicant demonstrates to the satisfaction of the State Department of Health Services that the methods of treatment and reliability features will assure an equal degree of treatment and reliability.

7 The Garay Reclamation Facility treatment process is at least equivalent to these.

Source: California Administrative Code, Title 22, Division 4.

Markets served:

Nursery	Vineyards	Flower Seed Propagation
Forage	Sugar Beets	Golf Course

Responsibilities of the City and Water Districts:

The agreement between the three cooperating agencies provides for project construction and operation and maintenance. The project was constructed by the Santa Clara Valley Water District. The District will continue general administration of the project and provide for billings and collections for water deliveries. The City of Gilroy will operate and maintain the oxidation pond, the chlorination facilities, and the pumping station located at their sewage treatment plant. The South Santa Clara Valley Water Conservation District will operate the pipeline distribution system, see to the delivery of water to the users and read delivery meters.

Cost and Financing:

Total project cost was \$1.88 million, and most financing was provided by a loan from the U. S. Bureau of Reclamation (USBR). Availability of a loan came with passage by Congress of the Emergency Drought Act of 1977. The USBR was contacted by local agencies and reacted favorably to a suggestion that a water reclamation project in Gilroy would indeed produce a supplemental water supply during this emergency period. Accordingly, a contract with the USBR was completed under which a \$1.25 million loan was made available. The loan application was submitted in April, 1977 and approved in May. A cooperative agreement between the City of Gilroy, the South Santa Clara Valley Water Conservation District, and the Santa Clara Valley Water District was completed in June. Payback of the USBR loan and other costs will be through sale of the reclaimed water and contributions from the city and the two water districts.

Cost of treatment facilities:

Lagoon, pumps, strainer, chlorination system	\$930,000
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Cost of distribution system:

Pipeline, valves, telemetry	\$950,000
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Total cost:	\$1,880,000
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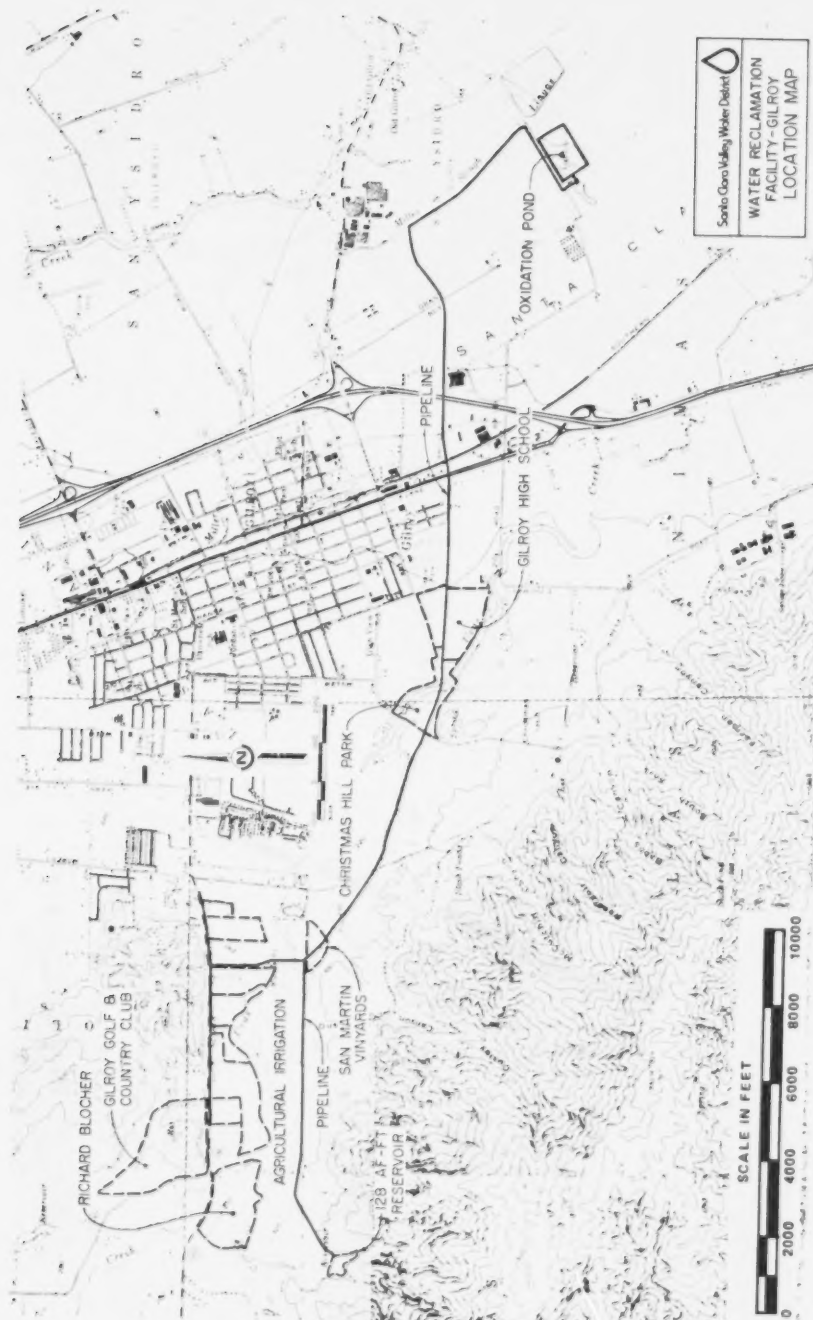
Estimated annual operation and maintenance cost:	\$ 100,000
--	------------

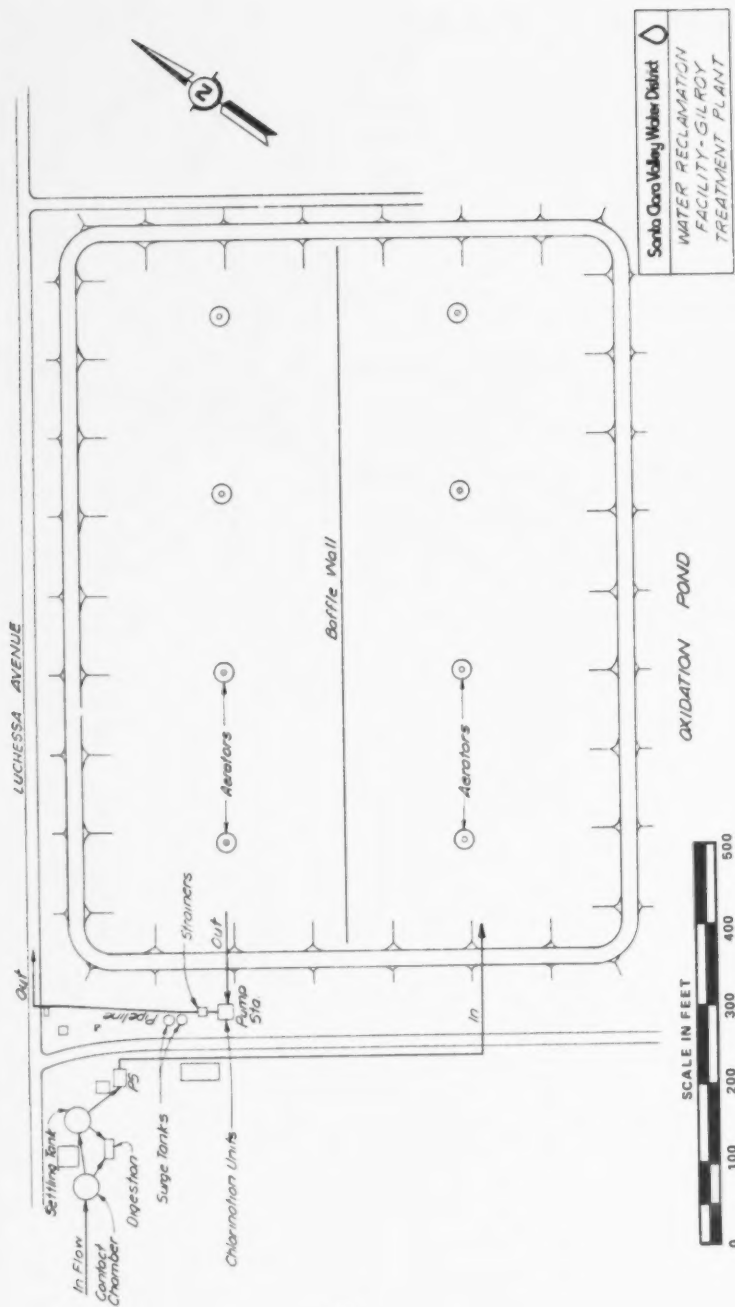
How much will users pay?

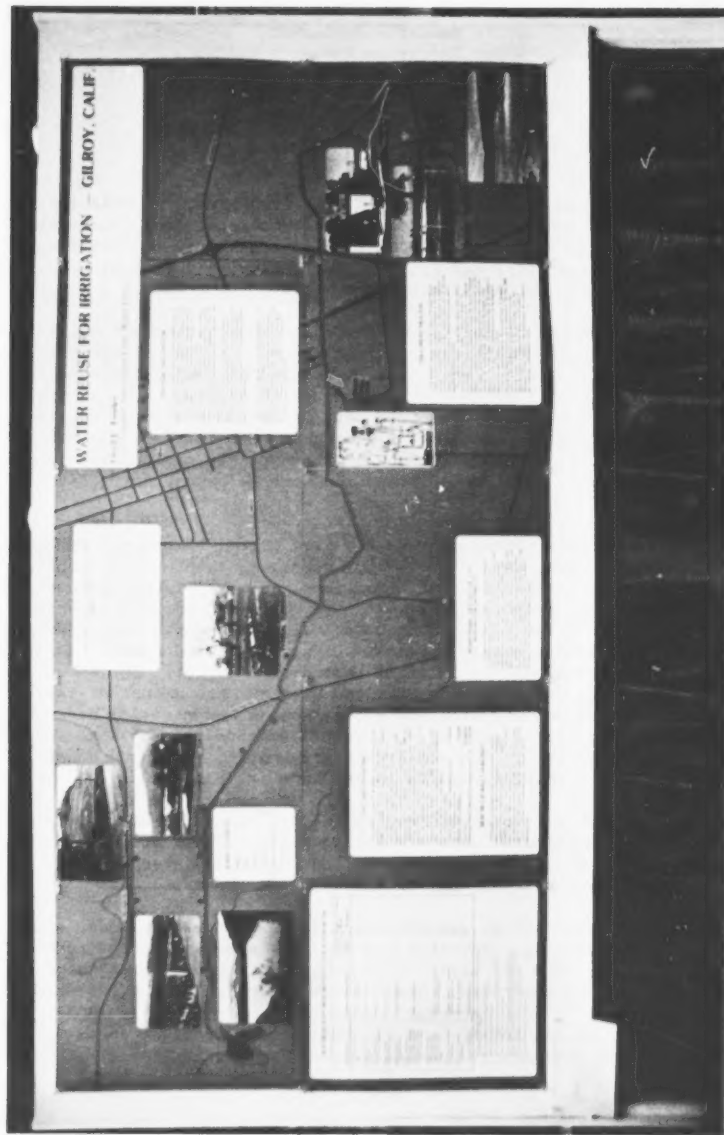
All markets are furnished water under specials contracts which provide for the delivery and payment of a minimum quantity of water. Additional quantities can be supplied if available. The contracts are for a term of 20 years. The price of water is \$8 per 1,000 cubic metres for water used for agricultural purposes and \$32 per 1,000 cubic metres for water used for municipal purposes (other than agricultural uses). The contracts provide for escalations in price should costs of operation rise.











INCREASING WATER SUPPLY FOR HOME IRRIGATION

Barney P. Popkin
Research Assistant
Water Resources Research Center
The University of Arizona
Tucson, Arizona 85721

ABSTRACT

Low rainfall and humidity, and high evapotranspiration, make irrigation necessary for domestic plant growth and commercial crop production in the American Southwest. Irrigation is restricted to dwindling ground-water supplies, and rainfall- or snowmelt- dependent streamflow. Both sources are susceptible to droughts and overdrafts.

A large percentage of potable water used in Southwestern homes is used for irrigation of domestic gardens and yards. Another large percentage is returned to sewers. Water fees increase with dwindling supplies, increasing depths to ground water, scarce surface water, and high treatment costs. Sewer-user fees increase because of high construction and treatment costs. Both water and sewer fees increase because of rapid urban expansion and increased water-quality standards. As fees increase, supplemental home irrigation sources become attractive and are sought.

Major supplemental water sources are grey water, harvested runoff, and roof runoff. Grey water is all home sewage effluent other than from toilets; it includes effluent from sinks, showers, tubs, dishwashers, clotheswashers, swimming pools, and evaporative coolers. Harvested runoff is all water produced from specially landscaped or prepared surfaces; such as driveways, streets, walks, or treated land. Roof runoff is all water produced by rainfall that drains a roof. The amount of grey water depends on family size and habits. The amount of harvested runoff depends on land size and slope, soil's and material's properties, and rainfall. The amount of roof runoff depends on roof size and geometry, and rainfall. The quality of grey water, harvested runoff, and roof runoff is generally suitable for home irrigation.

Engineering systems are required to collect, store, treat, distribute, and apply supplemental home irrigation water. There are many possible combinations of components, but the most preferred systems will have low capital expenditure and low energy requirements.

Economic considerations are different for large and small scales. Large-scale analysis indicates that a large and significant reduction in municipal costs and services is possible if supplemental home irrigation water is developed. Small-scale analysis for the 100-unit trailer park, 50-unit two-story apartment complex, double family home of six, and single family home of three, indicates that costs are favorable for supplemental irrigation systems, particularly for future large-scale units.

A suggested research program emphasizes field trials and demonstrations, which test design, operation, maintenance, and economics, as well as public and institutional acceptance.

INTRODUCTION

This paper is a written summary of a poster presentation prepared for the American Water Works Association's Water Reuse Symposium. The essential poster items, with additional items, are presented as figures to this text, submitted in a logical presentation order. Because the figures are self-explanatory, each is not reviewed in the text, unless additional clarification, definition, or expansion is required.

DEFINITIONS AND WATER USE

Figures 1 to 4 give the title, objective, supplemental water sources and definitions. Figures 5 to 7 give domestic water and wastewater load for a Southwestern family, and estimated water withdrawals, consumptive use, and sewage loads, and estimated reusable domestic wastewater for the Southwestern United States. Some of this information is adapted from published sources (1,2,3).

AVAILABLE WATER

Figures 8 to 11 show the depth of weekly irrigation possible for typical family's grey water, grey water available for different populations, irrigation available from roof runoff, and irrigation areas possible for various applied water volumes. Figures 10 and 11 can be used to estimate the contribution from harvested runoff, if the percentage of rainfall which can be harvested from treated land surface is estimated (4). The University of Arizona (3) produced a useful booklet to help select outdoor plants with low and moderate water requirements. See (5) for more information.

WATER QUALITY

Figure 12 summarizes water-quality characteristics of selected grey water. Harvested runoff will be low in salts, but possibly high in suspended solids and turbidity. Roof runoff will have the best quality, though it may show some turbidity. Figure 13 summarizes water-quality treatments. Note that a combination of crop filtration and uptake, dilution, filtration, soil filtration and uptake, and storage account for all listed quality variables. Water-quality aspects are reviewed in (6).

ENGINEERING SYSTEMS

These are summarized in Figure 14. The Farallones Institute (7) and others (8,9) give some practical designs.

ECONOMICS

These may be viewed as large scale and small scale.

Large Scale

If potable water usage can be reduced by reducing irrigation use of potable water, a savings in municipal services is achieved. If 65 percent of the used potable water is for home irrigation, it might represent up to 65 percent of the water bill. That amount of water, and its charge, can be reduced by 68 to 100 percent, depending on use and rate structure. If much of this reduction is due to grey-water reuse, sewer-use fees also could be reduced. And, if much of this

reduction is due to harvested and roof runoff, then street flooding and storm-runoff costs may be reduced. A large reduction in municipal costs and services is possible if supplemental home irrigation water is developed, and water-use and sewer-fee costs are commensurate with water and sewage reductions. Municipalities could save money from savings in water supply, sewage treatment, and flood damages.

Small Scale

Figures 15 to 17 show minimum repayment periods for non-integrated and integrated seven-day storage systems, and integrated three-day storage systems. These are for four illustrative cases, and two water sources, with ten adjustments. They assume that each trailer has a 12- by 60-ft roof, that the apartment complex has a 6,250 sq ft roof, and that the double and single family homes each have a 1,500 sq ft roof. They assume that the trailer park, apartment complex, and each home occupies 7.92, 0.41, and 0.18 acres of land, and that 12, 40, and 4 percent of each is paved. It's assumed that a water-harvest efficiency of 50 percent applies to unpaved areas. Rainfall is assumed to be ten inches a year.

The value of water is based on current Tucson rates from the author's bills, or \$36 per person per year using 208 gpcd, or \$3.56 per 1,000 cu ft. Grey water and laundry effluent have an added value in reducing sewer-use fees in this calculation, or \$31 per person per year for 100 gpcd, or \$6.38 per 1,000 cu ft. The unspecified value in reducing street runoff, which may cause erosion, street repairs, and flooding, is not computed. In short, grey water and laundry effluent are worth \$9.94 per 1,000 cu ft, and harvested and roof runoff are worth only \$3.56 per 1,000 cu ft in this exercise.

The calculations also assume that irrigation will occur otherwise exclusively with fresh water. All repayment calculations allow a ten-percent mortgage interest.

Table 1 shows comparative costs of supplemental water. Comparative unit costs for supplemental water systems indicate that:

1. The unit cost for all sources increases with decreasing number of people, and land and roof size.
2. The cheapest source of supplemental water is most grey water.
3. The most expensive source is harvested runoff, except where dispersed trailers produce more expensive roof runoff.
4. Integrated storage, and short-term storage, considerably reduce unit and total costs.

SUMMARY AND RESEARCH

Figure 18 summarizes the presentation. Additionally,

1. Municipalities often discourage supplemental water use by minimum charges for water use and sewer use. Tucson, Arizona, is such a case.
2. States often discourage supplemental water use by outlawing its application. Arizona is such a state (10).

3. The federal government has discouraged such water sources by heretofore inadequate research funds.

A research program is badly needed (11). It should:

1. Emphasize field trials and demonstrations.
2. Test design, operation, and maintenance.
3. Evaluate economics.
4. Address public and institutional acceptance.
5. Review legal problems.

ACKNOWLEDGMENT

I thank Sol D. Resnick for his support in this project. K. J. DeCook was also supportive. C. B. Cluff provided useful conversation. M. Busse, S. Schuster, and N. Svacha drafted and typed this material.

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U.S. Dept. Interior, Wash., D.C., (1975), 85p.

TABLE 1

Comparison of Dollar Costs of Supplemental Water
For 1,000 Cubic Feet Over a Ten-Year Period

WATER SOURCE	TRAILER PARK	APARTMENT COMPLEX	DOUBLE FAMILY HOUSE	SINGLE FAMILY HOUSE	
A	8.44	8.90	30.50	48.73	Seven-day Nonintegrated Storage
B	18.72	24.37	77.95	103.37	
A	0.69	0.95	2.45	3.98	Seven-day Integrated Storage
B	1.41	2.19	7.10	7.74	
A	0.30	0.42	1.12	1.82	Three-day Integrated Storage
B	0.62	0.98	3.20	3.52	

A: Most grey water, and harvested and roof runoff.
B: Laundry effluent, and harvested and roof runoff.

INCREASING WATER SUPPLY FOR HOME IRRIGATION

Barney P. Popkin
Water Resources Research Center
University of Arizona
Tucson, Arizona 85721

FIGURE 1

OBJECTIVE

- Review Supplemental Water Sources
For Home Irrigation With Respect To
 - Water Availability (Quantity)
 - Water Quality
 - Engineering Systems
 - Economics

FIGURE 2

SUPPLEMENTAL WATER SOURCES

- Grey Water
- Harvested Runoff
- Roof Runoff

FIGURE 3

GREY WATER

- All Home Sewage Effluent Other Than Toilet Wastes. Effluent From Sinks, Clothes Washers, Swimming Pools, Evaporative Coolers, Showers, Tubs

HARVESTED RUNOFF

- Water Produced From Prepared Land Surfaces. Runoff From Driveways, Streets, Walks, Lawns.

ROOF RUNOFF

- Water Produced By Rainfall That Drains Off A Roof.

FIGURE 4

DOMESTIC WATER USE AND WASTEWATER LOAD FOR A SOUTHWESTERN FAMILY

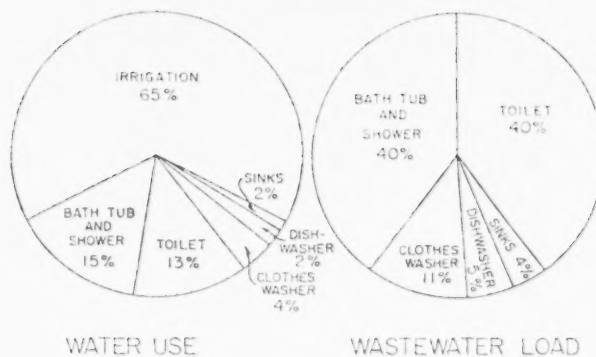


FIGURE 5

ESTIMATED WATER WITHDRAWALS, CONSUMPTIVE USE, AND SEWAGE LOADS FOR THE SOUTHWEST

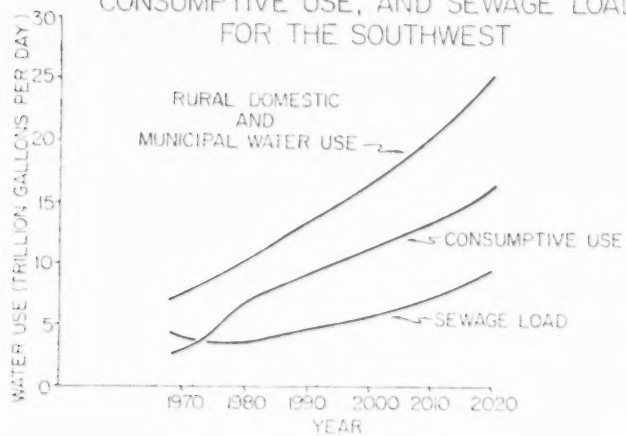


FIGURE 6

ESTIMATED QUANTITIES OF REUSABLE DOMESTIC WASTEWATER FOR THE SOUTHWEST



FIGURE 7

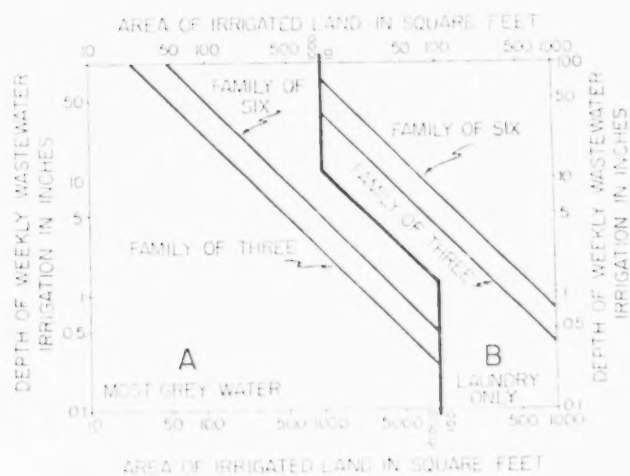


FIGURE 8

IRRIGATION WATER AVAILABLE FROM GREY WATER IN CUBIC FEET PER WEEK



FIGURE 9

IRRIGATION WATER AVAILABLE FROM ROOF RUNOFF IN 1000 CUBIC FEET

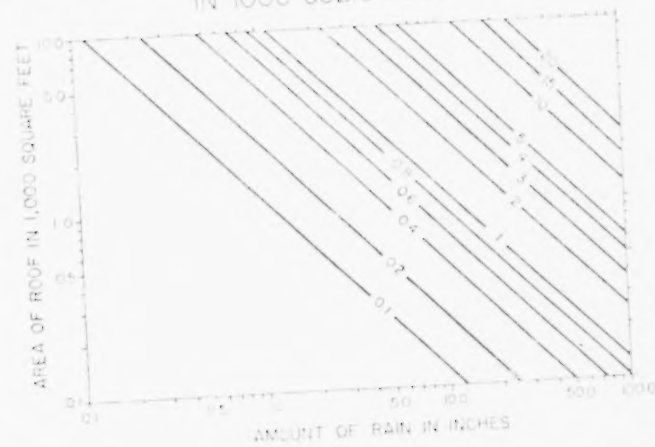


FIGURE 10

QUANTITY OF AVAILABLE IRRIGATION IN INCHES OF DEPTH

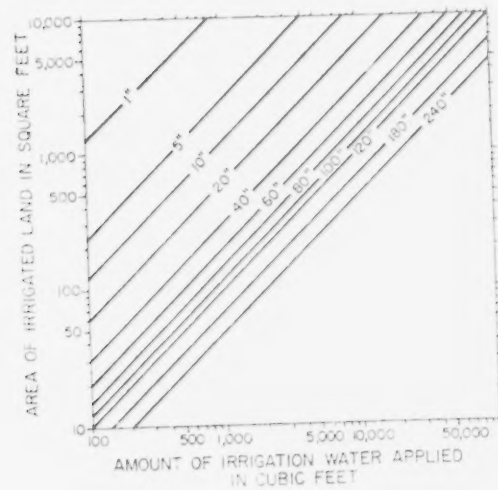


FIGURE 11

WATER SOURCE	CHARACTERISTIC													
	BACTERIA	BLEACH	CHLORINE	FOAM	FOOD PARTICLES	HAIR	HIGH pH	HOT WATER	NITRATE	COOR	OIL AND GREASE	ORGANIC MATTER	OXYGEN DEMAND	PHOSPHATE
AUTOMATIC CLOTHES WASHER		•		•				•	•	•	•	•	•	•
AUTOMATIC DISH WASHER	•			•	•			•	•	•	•	•	•	•
BATH TUB AND SHOWER	•					•		•		•	•	•		•
EVAPORATIVE COOLER													•	
SINKS, INCLUDING KITCHEN	•				•			•		•	•	•	•	•
SWIMMING POOL			•										•	•

WATER QUALITY CHARACTERISTICS OF SELECTED DOMESTIC WASTEWATER

TREATMENT	VARIABLE												
	BACTERIA	BLEACH	CHLORINE	FOAM	FOOD PARTICLES	HOT WATER	NITRATE	ODOR	OIL AND GREASE	ORGANIC MATTER	OXYGEN DEMAND	PH	PHOSPHATE
AERATION													
ALUM													
CARBON FILTRATION													
CHLORINATION													
CROP FILTRATION													
CROP UPTAKE													
DILUTION													
FILTRATION													
FLOTATION													
HYDROGEN PEROXIDE													
LIME													
SETTLING													
SOIL FILTRATION													
SOIL UPTAKE													
STORAGE													

TREATMENT FOR WATER-QUALITY VARIABLES

FIGURE 13

SUMMARY OF ENGINEERING REQUIREMENTS BY WASTEWATER SOURCE

WASTEWATER SOURCE	COLLECTION		STORAGE	TREATMENT	DISTRIBUTION
	TO DIRECT USE	TO STORAGE			
① BATHROOM SINK, BATHTUB, SHOWER	PLUMBING & PIPES		MINIMAL	MODERATE	PLUMBING
② EVAPORATIVE COOLER	HOSE OR PIPE, NOZZLE OR FAUCET	EASY PLUMBING, HOSE OR PIPES, OR SAME AS ①	TO EXTENSIVE MAY REQUIRE	LITTLE	OR HOSES
③ KITCHEN SINK, DISH WASHER	SAME AS ①		EXCAVATION, TUBS,	MODERATE	MAY
④ LAUNDRY CLOTHES WASHER	SAME AS ②		TANKS, OR	LITTLE	REQUIRE
⑤ SWIMMING POOL	SAME AS ②		PONDS	LITTLE	PUMPS

APPLICATION: FLOOD AND OVERLAND FLOW — LOWEST COST. REQUIRES LANDSCAPING FOR SLOPES, FURROWS OR FLOOD BASINS.

SPRINKLER — REQUIRES SPRINKLER HEADS AND PRESSURE.

TRICKLE — HIGHEST COST. REQUIRES FILTERS AND EMITTERS.

FIGURE 14

REPAYMENT PERIOD FOR NONINTEGRATED SEVEN-DAY STORAGE

CASE AND WATER SOURCE	NORMAL	(1) CUT LABOR COSTS IN HALF	(2) SECOND-HAND MATERIALS	(3) WATER-VALUE DOUBLES	(1) AND (2)	(1) AND (3)	(2) AND (3)	(1),(2) AND (3)	(4) WATER VALUE TRIPLES	(1),(2) AND (4)
1A	*	23	15	7	12	6	5	5	4	3
1B	**	**	**	**	**	**	**	*	*	12
2A	25	19	13	7	11	6	5	4	4	3
2B	**	**	**	**	**	**	**	25	24	10
3A	**	**	**	**	**	**	**	**	**	14
3B	**	**	**	**	**	**	**	**	**	**
4A	**	**	**	**	**	**	**	**	**	**
4B	**	**	**	**	**	**	**	**	**	**

NOTES: CASES, 1 = 100-UNIT TRAILER PARK WITH FIVE COMMON LAUNDRIES
2 = 50-UNIT TWO-STORY APARTMENT COMPLEX WITH FOUR COMMON LAUNDRIES
3 = DOUBLE FAMILY HOME OF SIX
4 = SINGLE FAMILY HOME OF THREE

WATER SOURCES, A = MOST GREY WATER, ROOF RUNOFF, HARVESTED RUNOFF
B = LAUNDRY EFFLUENT, ROOF RUNOFF, HARVESTED RUNOFF

* REPAYABLE IN MORE THAN 25 YEARS.
** NOT FINANCEABLE, AS MONTHLY VALUE IS SMALLER THAN INTEREST

FIGURE 15

REPAYMENT PERIOD FOR INTEGRATED SEVEN-DAY STORAGE

CASE AND WATER SOURCE	NORMAL	(1) CUT LABOR COSTS IN HALF	(2) SECOND-HAND MATERIALS	(3) WATER-VALUE DOUBLES	(1) AND (2)	(1) AND (3)	(2) AND (3)	(1),(2) AND (3)	(4) WATER VALUE TRIPLES	(1),(2) AND (4)
1A	9	9	7	4	6	4	3	3	3	2
1B	**	**	**	18	**	15	11	9	9	6
2A	14	12	9	5	8	4	4	4	3	2
2B	**	**	**	25	**	17	14	10	10	6
3A	**	**	**	**	**	19	21	12	12	7
3B	**	**	**	**	**	**	**	**	**	**
4A	**	**	**	**	**	**	**	**	**	17
4B	**	**	**	**	**	**	**	**	**	**

NOTES: CASES, 1 = 100-UNIT TRAILER PARK WITH FIVE COMMON LAUNDRIES
2 = 50-UNIT TWO-STORY APARTMENT COMPLEX WITH FOUR COMMON LAUNDRIES
3 = DOUBLE FAMILY HOME OF SIX
4 = SINGLE FAMILY HOME OF THREE

WATER SOURCES, A = MOST GREY WATER, ROOF RUNOFF, HARVESTED RUNOFF
B = LAUNDRY EFFLUENT, ROOF RUNOFF, HARVESTED RUNOFF

** NOT FINANCEABLE, AS MONTHLY VALUE IS SMALLER THAN INTEREST.

FIGURE 16

REPAYMENT PERIOD FOR INTEGRATED THREE-DAY STORAGE

CASE AND WATER SOURCE	NORMAL	(1) CUT LABOR COSTS IN HALF	(2) SECOND-HAND MATERIALS	(3) WATER-VALUE DOUBLES	(1) AND (2)	(1) AND (3)	(2) AND (3)	(1),(2) AND (3)	(4) WATER-VALUE TRIPLES	(1),(2) AND (4)
1A	3	3	3	2	3	2	2	1	1	1
1B	13	12	9	5	8	5	4	3	3	2
2A	4	4	3	2	3	2	2	2	2	1
2B	17	12	12	6	7	5	5	3	4	2
3A	*	14	16	7	10	5	6	4	4	3
3B	**	**	**	**	**	**	**	**	**	17
4A	**	**	**	19	**	11	13	8	9	5
4B	**	**	**	**	**	**	**	**	**	**

NOTES: CASES. 1= 100-UNIT TRAILER PARK WITH FIVE COMMON LAUNDRIES
2= 50-UNIT TWO-STORY APARTMENT COMPLEX WITH FOUR COMMON LAUNDRIES
3= DOUBLE FAMILY HOME OF SIX
4= SINGLE FAMILY HOME OF THREE

WATER SOURCES. A= MOST GREY WATER, ROOF RUNOFF, HARVESTED RUNOFF
B= LAUNDRY EFFLUENT, ROOF RUNOFF, HARVESTED RUNOFF

* REPAYABLE IN MORE THAN 25 YEARS
** NOT FINANCEABLE, AS MONTHLY VALUE IS SMALLER THAN INTEREST

FIGURE 17

SUMMARY

- Grey Water, Harvested Runoff, And Roof Runoff Are Supplemental Water Sources For Home Irrigation In The Southwest.
- Needed Engineering Systems Can Be Flexible And Innovative.
- Economics Are Favorable For Future Large-Scale Supplemental Water Systems
- Economics Become More Favorable As Water-Use And Sewage-Fee Charges Increase.
- Costs Can Be Reduced Considerably By
 - Providing Three-Day Water Storage
 - Integrating Collection, Storage, And Distribution Facilities.
 - Using Second-Hand Materials.
 - Exploiting Home-Owner Labor.

FIGURE 18
2081

RECLAIMED WASTEWATER AS A FEASIBLE WATER RESOURCE
FOR LANDSCAPE AND ORCHARD IRRIGATION

William R. Everest
Principal Engineer
PRC Toups Corporation, Consulting Engineer
Orange, California

Robert A. Paul
General Manager and Chief Engineer
Goleta County Water District
Goleta, California

Goleta, California, a water short area on the South Coast of Santa Barbara County is the site for a major wastewater reclamation program. The demand for water in the Goleta area has been in excess of safe yield of available water supplies for a number of years. As a result of this demand, the Goleta County Water District (GCWD) established a moratorium on new water system connections which has been in effect since 1972. As a possible alternative to alleviating this water supply problem, GCWD engaged PRC Toups Corporation to determine the potential and feasibility of reclaiming wastewater. This paper will discuss and summarize the findings and results of that study.

STUDY AREA CHARACTERISTICS

The study area depicted in Figure 1, is the service area of the GCWD. This area encompasses approximately 130 square miles or about 85,000 acres from the Pacific Ocean to Los Padres National Forest.

Water resources in the study area include surface waters originating in the Goleta Watershed and groundwaters of the Goleta Basin and its constituent sub-basins. Ten major creeks in the Goleta Watershed drain into the Goleta Slough between the University of California at Santa Barbara (UCSB) and the Santa Barbara Municipal Airport (SBMA) from which drainage is carried to the ocean.

The groundwater basin is almost completely separated from water-bearing deposits in the ocean by an impervious coastal barrier. The basin has been divided into four sub-basins on the basis of differences in water levels. Groundwater is also found in the Ellwood-Gaviota area west of Goleta. Wells in that area are generally low yielding in comparison with the Goleta Basin. Historically, extractions have been limited to approximately 600 ac-ft/yr.

AVAILABLE WATER SUPPLIES

Water supplies of the study area are obtained from both Cachuma Reservoir and the Goleta Groundwater Basin. The available supply from these two sources will increase from the current level of 13,619 ac-ft/yr

to 14,510 ac-ft/yr in 1990 and beyond. GCWD is the major water purveyor within the study area, although a number of users obtain water from small private wells and from diversion of natural stream flow.

In general, both Cachuma Reservoir and the Goleta Basin groundwater are considered fairly high in mineral content. Groundwater supplies exceed 700 mg/l of total dissolved solids (TDS) while Cachuma water has an approximate TDS content of 525 mg/l. Both supplies are classified as very hard. Water imported from Cachuma Reservoir is treated at GCWD's Corona del Mar Water Treatment Facility prior to distribution. Well water is chlorinated as minimum treatment, and in some cases iron and manganese removal is provided.

The demand for water in the study area has exceeded the firm yield of available supplies for several years. The firm yield of available supplies to the GCWD service area has been exceeded by as much as 4,700 ac-ft/yr in recent years. GCWD has relied upon temporary supplies to meet demands in excess of its firm supplies. These temporary supplies include purchases of Cachuma Reservoir water from other water agencies served by Cachuma Reservoir and "borrowing" of Cachuma Reservoir supplies for future allotments.

As a result of the deficit, a moratorium on most types of new service connections has been in effect in the District since December 7, 1972. Water conservation efforts in the area have reduced the existing deficit to its existing estimated level of 1,500 ac-ft/yr. The existing hookup moratorium will continue to be in effect until that deficit is eliminated. The Santa Barbara County Water Agency has projected water supply requirements for the study area as part of an overall assessment of Santa Barbara County water needs. Projections assume a lifting of the existing hookup moratorium by 1980. Those projections indicate that demand within GCWD will increase to nearly 30,000 ac-ft/yr by the year 2000 from a total existing level of nearly 19,000 ac-ft/yr. Irrigated agriculture within the District is expected to double its existing demand to over 11,000 ac-ft/yr by the year 2000.

EXISTING WASTEWATER TREATMENT FACILITIES

The Goleta Waste Treatment Facility (GWTF) was originally constructed in 1951 at its present site. Treatment at that time included primary and secondary oxidation, anaerobic sludge digestion and air drying, and disinfection. Effluent was discharged to four oxidation ponds which overflowed to San Pedro Creek. In 1961, subsequent to University of California, Santa Barbara, Isle Vista Sanitary District; and Santa Barbara Municipal Airport joining the system, the oxidators were converted to primary sedimentation tanks, and a new headworks structure, comminutor aerated grit removal tank, biofilter, oxidation ponds, and sludge drying beds added. By 1965, an administration building and mile-long ocean outfall were constructed. At that time, the biofilter was taken out of service. A new influent pumping station, third primary clarifier, and second anaerobic digester were also added. In 1971, a second comminutor and aerated grit removal tank were added along with two additional

primary clarifiers, thereby increasing the facility's nominal capacity from 7 mgd and 14 mgd. A solids stabilization basin and improvements to the clarifier inlet baffles were constructed in 1975 as part of an interim improvement program. The existing treatment system consists of primary sedimentation with ocean disposal.

MARKET POTENTIAL AND DEMAND

The feasibility of using reclaimed wastewater is dependent upon several variables including: type of reuse, water quality criteria, governmental regulations, and location and amount of demand. In the Goleta area, wastewater could be reused for landscape and agricultural irrigation. Each of these uses is reviewed in the following sections and their potential for implementation analyzed.

Landscape Irrigation

Potential sites for landscape irrigation in the Goleta area include golf courses, parks, and highways. Specific sites were identified based on previous studies by PRC Troups Corporation, GCWD, and interviews; information supplied by the Santa Barbara County Parks Department, Isla Vista Recreation and Park District (IVRPD), and California Department of Transportation (Caltrans); and from color aerial photos of the study area taken in January 1978.

Potential sites were delineated including 33 parks and open spaces, 6 golf courses, UCSB, and state highways.

Yearly and seasonal irrigation requirements for golf courses within the GCWD service area were quantified in previous studies of the Goleta area (1). The results of that study indicate irrigation requirements are highly seasonal, with monthly rates varying from 0 to 225 percent of yearly average. Actual frequency of application averages about 3 times per week through the entire year and peaks at about 4-5 times per week during the drier summer months. Theoretical irrigation requirements for landscape irrigation at golf courses in the Goleta area have been estimated at 33.4 in/yr, without consideration of irrigation water quality (1). However, high levels of dissolved solids concentrations in irrigation waters require that additional irrigation water be applied to leach the salts from the root zone of the plants. Turf grasses utilized at golf courses in the Goleta area generally consist of a fairly salt tolerant species: bermuda grass, perennial rye, etc.

Utilizing guidelines published by the University of California, Cooperative Extension-Committee of Consultants, the District's existing water supply quality (TDS = 560 mg/l) requires leaching of about 3 percent of the theoretical irrigation requirement to result in no yield reduction of the turf grass (2). Utilization of wastewater of existing quality from the GWTF (TDS = 1330 mg/l) would increase the necessary leaching requirement to 6 percent of the theoretical irrigation requirement to maintain a zero yield reduction. Thus, total irrigation requirement of landscaping in the study area is assumed to be 106 percent of 33.4 in/yr,

or 35.4 in/yr. Therefore, for planning purposes, irrigation requirements for landscaping were based on a unit factor of 36 in/yr. Landscape irrigation requirements for highways are based on average historical uses. Peaking factors for all sites were assumed the same as those determined for the golf courses.

Landscape irrigation is generally accomplished by some form of sprinkler irrigation system, either by automatic solid-set sprinklers or rainbird hose-pull systems. Some potential sites were eliminated from further consideration due to the large number of interconnections between irrigation and domestic supply systems. Segregation of the two systems for those sites would be economically impractical. Only those sites which already have separate systems or have systems that could be readily segregated were considered.

Agricultural Irrigation

Commercial agriculture within the Goleta area consists primarily of avocado and citrus groves located along the narrow valleys of the Santa Ynez foothills plus lesser amounts of nurseries, alfalfa, truck crops, and other miscellaneous crops scattered throughout the coastal plain south of the foothills. Field investigations were conducted and aerial photos, taken in January 1978, reviewed to delineate potential sites that could utilize reclaimed wastewater. Meetings and workshops were conducted with representatives of the major growers, agricultural consultants, and farm advisors to set criteria to delineate potential sites. The established criteria include consideration of area, topography, possible future land use, irrigation requirements, water quality requirements, and management attitudes.

Potential Existing Demand. Based on the established criteria plus the utilization of previously gathered information by other agencies, a total of about 4,970 acres of existing avocado and citrus groves have been delineated as potential users for reclaimed wastewater. Urbanization pressures and site isolation eliminate some agriculture areas in the coastal plain plus some additional foothill sites as potential users. Of the selected groves, the distribution of avocados and citrus is about 50-50 with the different crops intermixed throughout the individual sites. However, with some exceptions, the tendency in new plantings in the area is towards avocados. Future groves are expected to be 75-80 percent avocados and 20-25 percent citrus.

The irrigation season for fruit groves in the South Coast area of Santa Barbara County usually extends from April to November, with differences according to climatic variations. Over 80 percent of the water demand occurs during this period. Figure 2 illustrates typical seasonal variation in study area irrigation requirements based on actual water sales records maintained by GCWD. As shown, peak demands during July reach 210 percent of average annual demand. Minimum demands generally occur during the period of late January through early April.

Average yearly water use for irrigation of citrus and avocados is 1.5 ac-ft/yr/gross ac. This figure is based on utilization of irrigation water with mineral quality equal to existing water supplies. Utilization of irrigation water with higher dissolved solids concentrations would necessitate increased irrigation requirements to allow for leaching of salts from the root zone of the irrigated crops.

The 4,970 acres previously noted could potentially utilize an average of approximately 7,500 ac-ft/yr of reclaimed wastewater, provided no other water supply sources are utilized. However, during the period 1971-1976, water purchases from GCWD for irrigation at these sites have averaged just over 2,300 ac-ft/yr. This discrepancy is due to several factors; there are sites which are not within GCWD and therefore do not receive supplies from GCWD. The existing sites also receive irrigation water collected by private wells and from small private reservoirs which capture local runoff and do not rely solely on GCWD for irrigation water. Also, plantings have increased rapidly during the 1970's. Young avocado trees, when first planted, receive as little as 10 percent of the irrigation water mature trees receive, and will not receive the same amount of irrigation water until they are approximately 7 years old (3). Therefore, the irrigation demand of recently planted groves is considerably less than that of mature groves. It should be noted that purchases from GCWD at these sites has increased in recent years to as much as 3,270 ac-ft in 1976. As the new groves mature, total irrigation demand at the various sites can be expected to reach 7,500 ac-ft/yr.

Additional Future Agricultural Demand. The Goleta area has recently experienced considerable growth in agriculture, primarily in the form of avocado groves. This trend is expected to continue in the future, provided sufficient economical water supplies are available. In 1976, the University of California, Cooperative Extension Office in Santa Barbara County conducted a survey of landowners in the Goleta area, both within and outside the GCWD service area, in an effort to predict the most probable location and acreages for future citrus and avocado plantings in the study area. The results of the survey showed a total of nearly 7,500 additional acres could conceivably be planted. These future plantings, at maturity, would have a potential demand of over 11,300 ac-ft/yr of reclaimed wastewater provided no other water source is utilized for irrigation.

Current Irrigation Practices. The vast majority of older (5 years) groves employ overhead or low level sprinkler systems for irrigation. Drip systems are being utilized on nearly all the newer farms, especially in the Goleta West area. Based on information obtained at the user participation workshops conducted as part of this study, drip irrigation will generally be used on new groves, but for economic reasons, will not replace existing sprinkler systems. Those farms currently using drip irrigation are required to filter their water supply prior to distribution, regardless of the source of supply. This is to prevent plugging of the small apertures characteristic of the drip system.

Existing irrigation systems and domestic supply lines are generally interconnected at the agricultural sites. However, because domestic service is supplied by a minimum number of on-site water lines, minor problems, if any, would be encountered in eliminating any potential cross connections between on-site domestic and irrigation distribution systems.

WATER QUALITY REQUIREMENTS

Wastewater must be disposed or reused in a manner which will protect the public health, maintain receiving water quality and prevent adverse effects at the site of application or disposal. These conditions, along with economic considerations, determine the degree and type of wastewater treatment necessary prior to reuse or disposal.

Regulatory Requirements

Water quality objectives for the study area were established by the State of California primarily through the Regional Water Quality Control Board (RWQCB), Central Coast Region and the State Department of Health Services (CDH). The RWQCB established water quality objectives for both surface waters and groundwaters in the area as well as establishing specific waste discharge requirements for point source discharges. CDH establishes regulations and standards for the use of reclaimed wastewater and protection of public health. The standards include required levels of treatment and specific bacteriological criteria for various types of reuse. A summary of these standards is indicated in Table 1. As shown, the requirements vary considerably for the type of reuse and method of application.

Water Quality Requirements For Irrigation

One of the most important considerations in evaluating the reuse potential of wastewater for irrigation are water quality requirements. In addition to the requirements established by CDH, quality requirements are determined by an evaluation of the possible adverse effects on the irrigated crop by chemical constituents in the water and also by the potential problems of clogging the irrigation systems, particularly drip application systems. Whereas the CDH regulations specify distinct levels of quality in terms of bacteriological and physical parameters, the specification of non-injurious constituents concentrations to crops and irrigation systems is a difficult and involved task requiring an extensive review and evaluation of available literature and other data prepared and compiled by numerous researchers and experts in agronomy.

Mineral Constituents. In considering the potential for reusing wastewater for irrigation, it is necessary to consider the effects of the specific chemical constituents of the water and their relation with the soil and plant metabolism. Extensive studies have been conducted in efforts to determine specific acceptable water quality criteria for irrigation waters. Probably the most encompassing attempt to determine water quality criteria for agriculture has been conducted by the University of California, Cooperative Extension, Committee of Consultants.

The results of their analyses have been published in "Water Quality Guidelines for Interpretation of Water Quality for Agriculture" (2). These guidelines, summarized in Table 2 are intended for use in estimating the potential hazards to crop production associated with long-term use of the particular water being evaluated.

Although the UC guidelines are quite detailed and extensive, they are considered to be "flexible." As stated by the authors of the guidelines, "The guidelines should be modified when warranted by local experience and special conditions of crop, soil, method of irrigation or level of soil-water-crop management. Changes of 10 to 20 percent above or below an individual guideline value may have little significance if considered in proper perspective along with other variables that enter into a yield of crop" (2).

As a result, a detailed literature search was conducted along with interviews with agricultural experts; including those at UC Riverside who developed the Cooperative Extension Guidelines, the UC Cooperative Extension-Office in Santa Barbara County, and the Fruit Growers Lab in Santa Paula; in order to establish irrigation quality criteria specific to the study area. The criteria take into account numerous parameters characteristic of the Goleta area such as sensitivity of crops to chemical constituents, irrigation efficiency, and the physical and chemical characteristics of the soil. The following sections briefly describe particular chemical constituents of significance in irrigation waters and their impacts on crops.

Salinity. The effect of total dissolved solids or salinity is one of the most important of agricultural water quality considerations. The mechanism of effect of salinity is the change in osmotic pressure of the soil solution, which is related to the availability of water for plant consumption. Plants have been observed to wilt in fields apparently having adequate water content. This is usually the result of high soil salinity creating a physiological drought condition. Specifically, the ability of a plant to extract water from a soil is determined by the following relationship:

$$TSS=MS+SS$$

In this equation, the total soil suction (TSS) represents the force with which water in the soil is withheld from plant uptake. In simplified form this factor is the sum of the matric suction (MS), or the physical attraction of soil for water, and the solute suction (SS), or the osmotic pressure of the soil water.

As the water content of the soil decreases due to evapotranspiration, the water film surrounding the soil particles become thinner and the remaining water is held with increasingly greater force (MS). Since only pure water is lost to the atmosphere during evapotranspiration, the salt concentration of soil solution (SS) increases rapidly during

the drying process. Since the matric suction of a soil increases exponentially on drying, the combined effects of these two factors can produce critical conditions with regard to soil water availability.

The dissolved solids or saline content of the soil solution results from natural dissolution of soils minerals and primarily from that added as irrigation water or fertilizers. Water moves downward primarily through gravitational and capillary forces until it approaches a state where further movement is slow; then moves back toward the surface as a result of evapotranspiration. With adequate leaching, however, excess water passes through the root zone carrying the salt downward. Soil salinity in the root zone will vary between irrigations, but may, under certain circumstances, present a stable pattern over long periods of time.

Plant species vary in their tolerance to increasing salinity concentrations. Crops with relatively low tolerance levels include most fruit crops, including avocados, citrus, and berries (2). Plant growth limitation is progressive as salt concentration increases, and the harmful effects of a high TDS-content irrigation water appear only after extended use. The deleterious effects of salinity on plants may be modified by many factors; such as evapotranspiration level, crop type, soil character, drainage conditions, predominate ions, and climate. The relative salt tolerance of specific crops reflects plant growth under typical agricultural management practices, and not in the inherent ability of crops to withstand salinity under conditions that are uniform for all crops.

Although TDS levels of less than 1,000 mg/l (EC less than 1,500 mhos/cm) is a recommended maximum for citrus and avocado irrigation, several existing sites in California are currently using water with higher levels, without adverse effects on crop yields. The Irvine Company uses reclaimed wastewater with 1,000 mg/l TDS for citrus irrigation. Fallbrook Sanitary District in San Diego County provides wastewater of 1,200 mg/l TDS for both avocado and citrus irrigation with no reported adverse effects (4). Orchards in the Santa Clara Valley in Ventura County have historically used water containing 1,500 mg/l TDS, again without significant detrimental effects (5). The Riverside County Farm Advisor has stated that for new avocado groves, "Depending on the type of irrigation used, we can get by with water with relatively high salt content. If drip irrigation is used, water as high as 1,200-1,500 ppm (mg/l) is acceptable, but with sprinklers, we can get no higher than 1,000-1,200 ppm because of possible damage to foliage" (6). In cases where excessive salinity exists in irrigation water, increased leaching can be used to flush out excess salts in the soil. Landscaping usually requires less stringent irrigation quality. Water quality criteria for grasses have been documented elsewhere, and since they are not overly stringent, generally established criteria are used for the Goleta area. Grasses commonly used for parks and golf courses in the Goleta area include bermuda, bluegrass, and rye grass. Based on studies at the U.S. Salinity Laboratory in Riverside, California, bluegrass has been shown to generally be the most sensitive of these

grasses to water quality. Within that system, bluegrass and rye salt tolerances are rated as "medium". Similarly, landscaping at parks and freeways includes natural growth plus plantings of groundcovers, shrubs, and trees. These plants are generally rated as being moderately tolerant to salinity (3).

It should be noted that, although high levels of dissolved solids will adversely affect crop growth, specific ions (e.g. boron, chloride, and sodium) may also cause extensive damage, even if the TDS concentration is otherwise below recommended levels. Because of this, individual ion concentrations must also be analysed to determine irrigation suitability.

Chloride. Chlorides are considered to be among the most troublesome anions in irrigation water. Although the accumulation of chloride in plant tissues exhibiting toxic symptoms is not an absolute indication of specific toxicity, chloride has been found to be definitely deleterious to citrus and avocados (7). Symptoms of chloride injury are a yellowing of leaves, followed by burning around their margins, and excessive exfoliation (laminated splitting). Leaves of the next growth are chlorotic but otherwise normal in appearance for several weeks, but later the previously mentioned symptoms manifest. This cycle may continue for many years without death of a plant. Plants grown in some soils show their injury not by obvious symptoms but by subnormal growth rates.

The Fruit Growers Laboratory, Inc. (FGL), in Santa Paula, Ventura County has done extensive work in Santa Barbara and Ventura Counties to specify deleterious concentration levels of ions for agricultural irrigation. The FGL has determined that chloride levels above 100 mg/l for avocados and 200 mg/l for citrus will cause significant yield decreases in those crops (5). As with salinity, landscaping is much more tolerant of chloride concentrations than citrus and avocados. Concentrations in excess 350 mg/l have been measured in wastewater used for landscape irrigation in San Buenaventura without any deleterious effects. Landscaping in the Goleta area can be expected to be at least as tolerant as that in San Buenaventura.

Boron. Plant species vary in boron requirements and in tolerance to excess boron; but, it is important to note that boron may cause damage even when present in very low concentrations. Crops most sensitive to boron are lemons, grapefruit, oranges, and avocados (7). Symptoms of boron damage may include burning of plant leaves, chlorosis, and necrosis, although these may not become apparent for several years. Citrus, avocados, and many other plants develop a burn on the tips or margins of mature leaves accompanied by chlorosis of interveinal tissue, that leads to premature leaf drop and reduced crop yield. At present, there is no chemical or soil to render boron non-toxic. However, growers in some areas are learning to live with marginal boron and salinity by maintaining fertility levels slightly above the usual optimum and by irrigating a little more frequently than normal.

Boron toxicity in many areas is traceable to use of irrigation waters with boron concentrations greater than 1.0 mg/l. Levels of boron up

to 1.0 mg/l are usually recommended as satisfactory for most crops, although sensitive crops may show some injury at that level. The use of reclaimed wastewater with 0.8 mg/l is currently performed by the Irvine Company for citrus irrigation. Avocado and citrus groves in the Fallbrook and Santa Clara River Valley areas are currently being irrigated with levels up to 1.0 mg/l B without adverse effects (4) (5). Landscaping plants are, again, more highly tolerant of boron levels than citrus and avocado crops. Based on data published by UC Cooperative Extensive, landscaping in the Goleta area can be rated as tolerant to boron, the most tolerant rating under that system (3).

Sodium and Sodium Adsorption Ratio. Sodium in irrigation water may become a problem in the soil solution as a component of total salinity increasing the osmotic concentration, and as a specific source of injury to fruits. It is mainly a problem, however, due to its effect on soil structure, infiltration, and permeability rates. Since good drainage is essential for management of salinity in irrigation, good soil structure and permeability must be maintained. A high percentage of exchangeable sodium in a soil containing swelling-type clays results in a dispersed condition unfavorable for water movement and plant growth.

The organic and clay fractions of the soil possess ion exchange properties. These fractions carry predominantly negative charges and, therefore, absorb positive ions (cations); predominantly calcium, magnesium, potassium, sodium, hydrogen, and aluminum. The distribution of adsorbed cations in the soil is in equilibrium with the soil solution. Anything that alters the composition of the soil solution, such as irrigation or fertilization, disturbs the equilibrium and alters the distribution of adsorbed ions in the soil. When calcium is the predominant cation adsorbed on this exchange complex, the soil tends to have a granular structure which is easily worked and readily permeable. When the amount of adsorbed sodium exceeds 10 to 15 percent of the total cations on the exchange complex, the clay becomes dispersed and slowly permeable unless a flocculated condition is maintained due to a high concentration of total salts. Where soils have a high exchangeable sodium content and are flocculated due to the presence of free salts in solution, subsequent removal of salts by leaching will cause sodium dispersal to occur unless leaching is accomplished using additions of calcium or calcium-producing amendments.

Adsorption of sodium from a given irrigation water is a function of the proportion of sodium to divalent cations (calcium and magnesium) in that water. The U.S. Salinity Laboratory proposed the sodium adsorption ratio (SAR) in order to estimate the degree to which sodium will be adsorbed by a soil from a given water when brought into equilibrium with it, SAR is determined by the following equation:

$$SAR = \frac{Na}{(1/2(Ca+Mg))}^{1/2}$$

where the sodium, calcium, and magnesium are expressed as equivalent concentrations. As soils tend to dry, the SAR value of the soil solution

increases even though the relative concentrations of the cations remain the same. This is apparent from the above equation where the denominator is a square-root function. This is a significant factor in estimating sodium effects on soils.

Heavy Metals. Most heavy metals are precipitated in the sludge of wastewater treatment facilities and are thus kept out of the effluent to be used for irrigation. However, some soluble heavy metal chelates are formed from the combination of heavy metals and organics in the wastewater effluent. The harmful effects on plants of heavy metals, such as arsenic, cadmium, lead, copper, etc., are the biggest unknowns in the use of reclaimed wastewater for irrigation. Heavy metals are held on soil cation exchange sites, but this does not constitute a permanent removal from the soil. The solubility of these heavy metal complexes varies with pH. Some plants, like clovers and grasses, will take up such metals as molybdenum, and concentrate those metals enough to be toxic to the animal that eats the plant, even though the plant may be healthy. Once heavy metals have reached toxic levels in the soil solution, they are difficult to remove. Many heavy metals are not removed by leaching because of their affinity for the soil, while tillage practices simply redistribute the heavy metals throughout the plow layer of the soil. The UC Cooperative Extension has developed recommended maximum concentrations of heavy metals in irrigation waters. Those recommendations are presented in Table 3.

Other Constituents. Clogging of irrigation systems can occur as a result of inorganic suspended solids, chemical precipitation, or by algal growth. Deposition of sediment not only reduces the capacity of irrigation systems to carry and distribute water, but can also decrease reservoir storage capacity. Suspended solids may cause undue wear on irrigation pumps and pipes and cause plugging of nozzles or drip applicators, reducing irrigation efficiency. Drip irrigation systems generally require filtration prior to distribution to reduce suspended solids even for treated water supplies. Generally, particulate matter that will pass a 200 mesh screen does not ordinarily cause plugging problems, even in drip systems. Also, encrustation of drippers with chemical precipitants seems to be less severe if the emitter is not open to the atmosphere (8). Algal problems can generally be controlled by chlorine dosage prior to distribution in the smaller diameter irrigation lines. Although potential problems can be severe due to these factors, quality criteria previously established for irrigation plus chlorination prior to distribution are sufficient to preclude potential problems associated with irrigation system cloggings in the study area.

Study Area Irrigation Water Quality Criteria

Presented in Table 4 are the water quality criteria developed for irrigation of agriculture and landscaping in the Goleta area. The indicated criteria are based primarily on a consensus of members of the UC Riverside and Santa Barbara County Cooperative Extensions, as determined at a meeting held between those individuals for the specific purpose of developing irrigation.

USER REQUIREMENTS

During the course of this study, several workshops were conducted with potential reclaimed wastewater users. The workshops were conducted to provide a forum wherein the potential users could express their concerns and provide input to the potential project in the planning stage. Through the workshops, it was determined that agricultural growers now using potable water supplied by GCWD would not accept reclaimed wastewater in lieu of potable water unless the reclaimed wastewater was a comparable quality (and cost) to their current irrigation water. As a result, it has been determined that a comparable quality of reclaimed wastewater must be provided.

Table 5 presents the anticipated mineral quality for Goleta wastewater flow receiving 100, 75 and 50 percent reverse osmosis treatment along with a comparison of Cachuma Reservoir water currently used for agricultural irrigation. Analysis of Table 5 shows that a treated wastewater blend receiving 75 percent treatment by reverse osmosis will produce an irrigation water of comparable quality to that of Cachuma Reservoir water. A blend receiving 50 percent treatment by reverse osmosis would produce an irrigation water considerably higher in mineral content than Cachuma water.

Comparison with agricultural irrigation criteria shows that the 75 percent blend is in the lower end of the "increasing problems" range whereas the 50 percent blend is in the middle to upper end of the "increasing problems" range and in the "severe problem" range for chloride. Further analysis of irrigation water characteristics can be made by determining theoretical effects on crop yields.

Presented in Table 6 are data on yield reductions and leaching requirements based on guidelines established by the UC Cooperative Extension. The data indicates that wastewater receiving 75 percent reverse osmosis treatment will result in zero reduction of avocados and lemons with the approximate same leaching requirements as needed for use of Cachuma water. A 50 percent treated wastewater will result in an estimated 7.5 percent yield reduction and an increased leaching requirement to a level of 10 percent.

Therefore, it was determined that reclaimed wastewater for agricultural irrigation must receive 75 treatment by reverse osmosis to insure the implementation of a reclamation project for irrigation of existing agriculture and to free-up potable water by substitution. Reclaimed wastewater receiving that level of desalting by reverse osmosis will be comparable in quality to Cachuma Reservoir water (and better than local groundwater) and, based on guidelines established by UC Cooperative Extension, not result in any field reductions or increased leaching requirements.

RECOMMENDED PLAN

Based upon a careful evaluation of the study area characteristics, market potential and treatment requirements a two-phased reclamation

project was recommended. The proposed project will consist of reclaiming wastewater for landscape irrigation of existing sites (Phase I) and agricultural irrigation of existing and future sites (Phase II). The total project will eventually reclaim approximately 8 mgd of wastewater. Phase I has an immediate demand for 965 ac-ft/yr while Phase II has the potential to utilize 7653 ac-ft/yr of reclaimed wastewater.

The treatment scheme to be provided includes secondary treatment by biofiltration and clarification. At this point that portion of flow to be used for landscape irrigation will be disinfected by chlorination prior to distribution. Effluent to be used in the agricultural phase will be further treated by chemical addition, rapid mixing, filtration and reverse osmosis to provide a suitable quality effluent for reuse. Effluent will be disinfected by chlorination and distributed. Separate distribution systems will be provided because of the different water quality requirements for each phase. Total projects cost are summarized in Table 7.

PILOT PLANT DEMONSTRATION

A short-term demonstration program is also being conducted as part of the study to satisfy the following needs:

- o Obtain grower acceptance and firm intent to utilize reclaimed water.
- o Develop detailed performance data for desalination of reclaimed water.
- o Refine desalination operating costs.
- o Investigate potential plugging of drip irrigation systems using reclaimed water.

Development and implementation of a viable water reclamation program in the Goleta area is highly dependent upon project acceptance by the potential users of the reclaimed water. The principal potential users have been identified as irrigators of existing and projected citrus and avocado orchards. Through the user participation program grower's concerns were identified. These concerns included: effects of reclaimed water mineral quality, impact of local conditions such as soils, and long-term use productivity yields. Concerns such as long-term crop yields would not be addressed in this demonstration, since 2 to 5 year's testing is required for meaningful results. However, other grower concerns such as water quality and local conditions would be addressed by the pilot plant demonstration.

Because of the sensitivity to certain salts of the crops proposed for irrigation by this reuse project, there is also a need to develop detailed performance data for removal of particular water quality parameters through the membrane desalination process. In addition to the conventional mineral parameters, these include: boron, cadmium, and nitrates.

The goal is to maximize rejection of the most restrictive mineral quality parameters, except nitrates. Nitrate rejection should be minimized to maintain the fertilizer value of the reclaimed water. In addition, the demonstration program will result in refined operating costs of the desalination process, particularly the cost of membrane replacement. Membrane life is affected by the particular quality of the local reclaimed water, and the desalination pretreatment processes.

The demonstration program would also provide input regarding the potential of plugging in drip irrigation systems used in the Goleta area, a key factor which needs to be resolved prior to implementation.

CONCLUSION

On March 6, 1979, the GCWD held an advisory election to determine the public's acceptance of wastewater reclamation and their willingness to continue with the reclamation project. With approximately 46 percent voter turnout, 64 percent of the people voted to proceed with the project and have the GCWD look into means of funding the local share of the project.

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TABLE 1. CDH WASTEWATER RECLAMATION CRITERIA FOR WASTEWATER

Reuse	Required Treatment (a)	Allowable Coliforms (b) (MPN/100 ml)
Food Crops		
Spray irrigation	Bio-oxidation, coagulation/clarification, filtration, disinfection	2.2 (d)
Surface irrigation		
General	Bio-oxidation, disinfection	2.2
Orchards and vineyards with no fruit contact with water or ground	Primary sedimentation	ns (e)
Exceptions	Considered on individual basis if crop undergoes pathogen destroying processing	ns
Fodder, Fiber & Seed Crops		
Fodder, fiber and seed crops	Primary sedimentation	ns
Pasture for milking animals	Bio-oxidation, disinfection	23
Landscape Irrigation		
Golf courses, cemeteries, freeway landscaping and landscaping	Bio-oxidation, disinfection	23 (f)
Parks, playgrounds and schoolyards (c)	Bio-oxidation, coagulation/clarification, filtration, disinfection	2.2 (d)
Recreational Impoundments		
Nonrestricted	Bio-oxidation, coagulation/clarification, filtration, disinfection	2.2 (d)
Restricted to non-bodily contact	Bio-oxidation, disinfection	2.2
Landscape-non-contact	Bio-oxidation, disinfection	23

(a) Primary effluent must not contain more than 0.5 milliliters per liter per hour of settleable solids. Filtration must provide an effluent with a turbidity that does not exceed an average of 2 turbidity units and does not exceed a maximum of 5 turbidity units. Alternative methods of treatment may be accepted if the applicant demonstrates to the satisfaction of CDH that they will assure an equal degree of treatment and reliability.

(b) Median as determined from results of last 7 days for which analyses have been completed.

(c) Proposed requirements will probably be adopted in 1978.

(d) Shall not exceed 23 MPN/100 ml in more than one sample within any 30-day period.

(e) Requirement not specified.

(f) Shall not exceed 240 MPN/100 ml in any sample.

TABLE 2. UC COOPERATIVE EXTENSION GUIDELINES FOR INTERPRETATION OF QUALITY OF WATER FOR IRRIGATION (a)

Problem and Related Constituent	No Problem	Increasing Problems	Severe
Salinity (b)			
EC of irrigation water, mho/cm	750	750-3000	3000
Permeability			
EC of irrigation water, mho/cm	500	500	2000
SAR (c)			
Specific ion toxicity (d)			
From root absorption			
Sodium (evaluate by SAR)	3	3 - 9	9
Chloride			
me/l	4	4 - 10	10
mg/l	142	142 - 355	355
Boron, mg/l	0.5	0.5 - 2.0	2.0 - 10
From foliar absorption (e)			
(sprinklers)			
Sodium			
me/l	3.0	3.0	--
mg/l	69	69	--
Chloride			
me/l	3.0	3.0	--
mg/l	106	106	--
Miscellaneous (f)			
NH ₄ - N,			
mg/l for sensitive crops	5	.05 - 30	30
NO ₃ - N,			
HCO ₃ (only with overhead			
sprinklers			
me/l	1.5	1.5 - 8.5	8.5
mg/l	90	90 - 520	520
pH	Normal range	6.5 - 8.4	--

- (a) Interpretations are based on possible effects of constituents on crops and/or soils. Guidelines are flexible and should be modified when warranted by local experience or special conditions of crop, soil, and method of irrigation (UC Cooperative Extension, 1975).
- (b) Assumes water for crop plus needed water for leaching requirement (LR) will be applied. Crops vary in tolerance to salinity. Refer to tables for crop tolerance and LR. The mmho/cm x 640 = approximate total dissolved solids (TDS) in mg/l or ppm; mmho x 1,000 = microhos.
- (c) SAR (sodium adsorption ratio) is calculated from a modified equation developed by U.S. Salinity Laboratory to include added effects of precipitation and dissolution of calcium in soils and related to CO₃ + HCO₃ concentrations.
- (d) Most tree crops and woody ornamentals are sensitive to sodium and chloride (use values shown). Most annual crops are not sensitive (use salinity tolerance tables). For boron sensitivity, refer to boron tolerance tables.
- (e) Leaf areas wet by sprinklers (rotating heads) may show a leaf burn due to sodium or chloride absorption under low humidity/high evaporation conditions. (Evaporation increases ion concentration in water films on leaves between rotations of sprinkler heads.)
- (f) Excess N may effect production or quality of certain crops; e.g., sugar beets, citrus, avocados, apricots, etc. (1 mg/l NO₃ - N = 2.72 lbs. N/acre foot of applied water.) HCO₃ with overhead sprinkler irrigation may cause a white carbonate deposit to form on fruit and leaves.

TABLE 3. RECOMMENDED MAXIMUM CONCENTRATIONS OF TRACE ELEMENTS IN IRRIGATION WATERS (UC COOPERATIVE EXTENSION 1975)

Constituent	Allowable Concentration (mg/l) (a)	
	For Waters Used Continuously on all Soil	For Use up to 20 Years on Fine Textured Soils of pH 6.0 TO 8.5
Aluminum (Al)	5.0	20.0
Arsenic (As)	0.10	2.0
Beryllium (Be)	0.10	0.5
Boron (B)	0.75	2.0
Cadmium (Cd)	0.01	0.05
Chromium (Cr)	0.10	1.0
Cobalt (Co)	0.05	5.0
Copper (Cu)	0.20	5.0
Fluoride (F)	1.0	15.0
Iron (Fe)	5.0	20.0
Lead (Pb)	5.0	10.0
Lithium (Li)	0.075 (b)	0.075 (b)
Manganese (Mn)	0.20	10.0
Molybdenum (Mo)	0.01	0.05 (c)
Nickel (Ni)	0.20	2.0
Selenium (Se)	0.02	0.02
Vanadium (V)	0.1	1.0
Zinc (Zn)	2.0	10.0

(a) These levels will normally not adversely affect plants or soils. No data is available for Mercury, Silver, Tin, Titanium, Tungsten.

(b) Recommended maximum concentration for irrigating citrus.

(c) For only acid fine-textured soils or acid soils with relatively high iron oxide contents.

TABLE 4. WATER QUALITY CRITERIA FOR IRRIGATION IN GOLETA

Irrigation Use	Parameter	Recommended Criteria		
		No Problems	Increasing Problems	Severe Problems
Agriculture (a)	Boron - mg/l	0.5	0.5 - 1.0	1.0
	Chloride - mg/l	75	75 - 140	140
	Salinity			
	Total Dissolved Solids - mg/l	500	500 - 1,000	1,000
	Electrical Conductivity mhos/cm	750	750 - 1,500	1,500
	Sodium Absorption Ratio			
	Plant Toxicity	4	4 - 8	8
	Soil Permeability	6	6 - 9	9
Landscaping (b)	Boron - mg/l	1.0	1.0 - 2.0	2.0
	Chloride - mg/l	200	200 - 400	400
	Salinity			
	Total Dissolved Solids - mg/l	1,600	1,600 - 2,600	2,600
	Electrical Conductivity mhos/cm	2,500	2,500 - 4,000	4,000
	Sodium Absorption Ratio			
	Plant Toxicity	8	8 - 12	12
	Soil Permeability	6	6 - 9	9

(a) Based on irrigation of lemons and avocados.

(b) Based on irrigation of grasses and ornamental landscaping.

TABLE 5. COMPARISON OF CACHUMA RESERVOIR AND DESALTED WASTEWATER MINERAL QUALITY

Constituent	Cachuma Reservoir Water	Concentration (a)		
		Treated Wastewater		
		100% by R.O.	75% by R.O.	50% by R.O.
Cations	-	15	22	28
Ammonium (NH ₄)	75	3	25	47
Calcium (C)	34	1	12	23
Magnesium (Mg)	4	4	8	12
Potassium (K)	47	40	99	158
Sodium (Na)	-	-	-	-
Anions	199	25	120	214
Bicarbonate (HCO ₃)	15	35	110	187
Chloride (Cl)	1.1	0.4	0.5	0.6
Fluoride (F)	0.1	0.1	0.2	0.3
Nitrate (NO ₃)	-	2	8	14
Phosphate (PO ₄)	234	20	94	168
Sulfate (SO ₄)	-	-	-	-
Boron (B)	0.3	0.5	0.6	0.7
Total Dissolved Solids (TDS)	523	150	445	740
Electrical Conductivity (EC)	945	240	720	1,200
Sodium Adsorption Ratio (SAR)	1	5	4	5

(a) Expressed as milligrams per liter except electrical conductivity which is expressed as micromhos per centimeter and sodium adsorption ratio which is expressed as units.

TABLE 6. THEORETICAL EFFECTS OF DESALTED WASTEWATER ON STUDY AREA AGRICULTURAL CROPS (a)

	Yield Reduction (%)	Leaching Requirement (%)
Irrigation Water		
Cachuma Reservoir	0	7.5
Reclaimed Wastewater		
100% reverse osmosis treatment	0	0
75% reverse osmosis treatment	0	7
50% reverse osmosis treatment	7.5	10

(a) Based on guidelines developed by the UC Cooperative Extension (1975) for determining theoretical yield reductions of crops as a function of electrical conductivity of irrigation water. Data is for avocados and lemons, whichever is more critical.

TABLE 7. SUMMARY COMPARISON OF DETAILED COST ESTIMATES OF MOST VIABLE ALTERNATIVES (ENR = 3500)

Project Phase	Reclaimed Wastewater (ac-ft/yr)(a)	Project Cost (\$1,000)	Annual Cost (\$1,000/yr)		Present Worth (\$1,000)(b)	Equivalent Unit Cost (\$/ac-ft)(c)		Unit Cost at Full Development (\$/ac-ft)	
			Capital Recovery	Operation and Maintenance (a)		Total (a)	Total (d)	Total (d)	Local (e)
Phase I Only	965	5,779	420	189	7,841	609	630	630	250
Phases I and II Combined	8,618	33,767	2,335	2,396	59,907	4,731	600	550	325

- (a) At full project development.
 (b) Present worth in 1983 based on initial project operation at that time. All project costs assumed to be incurred at beginning of project. Present worth of operation and maintenance costs are based on 6-5/8 percent interest over a 20-year period.
 (c) Sum of total annual costs over 20-year project divided by total volume of reclaimed wastewater over the same period.
 (d) Total of grant and non-grant funded facilities.
 (e) Takes into account grant funding by EPA and SARC8; based on local share of costs of project.

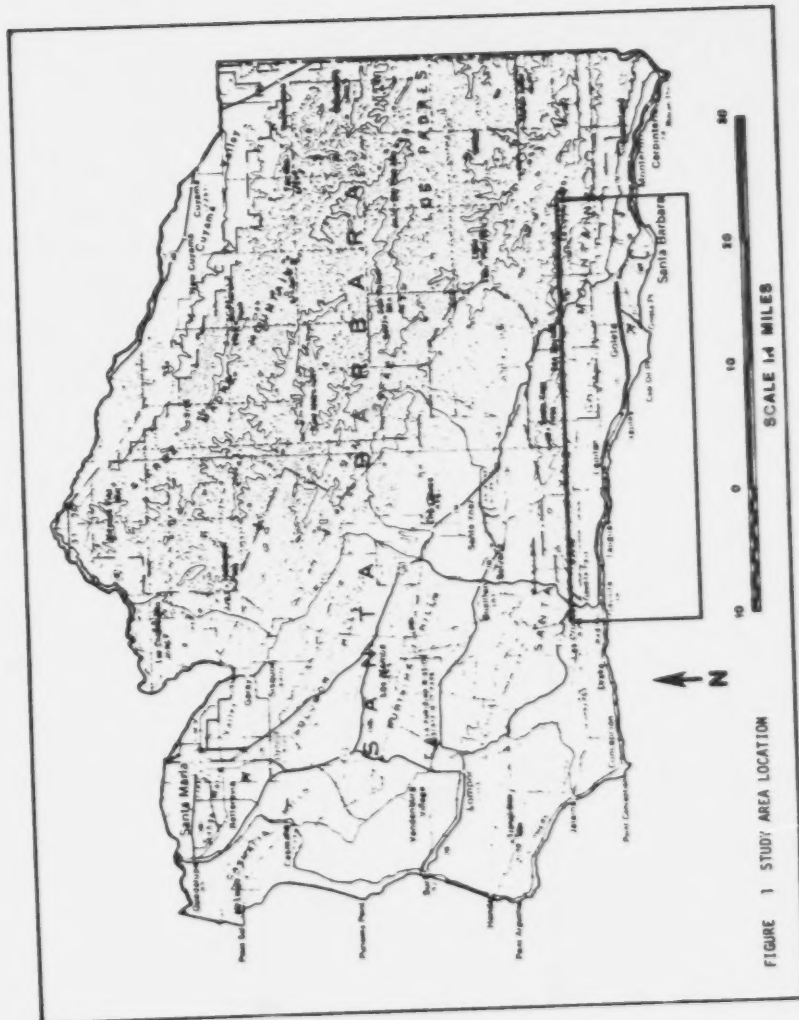


FIGURE 1 STUDY AREA LOCATION

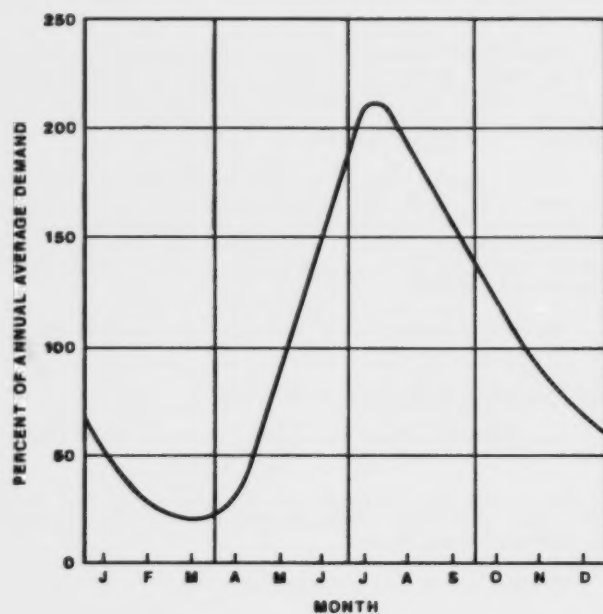


FIGURE 2 TYPICAL SEASONAL VARIATION IN STUDY AREA AGRICULTURAL IRRIGATION REQUIREMENTS

ACCELERATED SURFACE WATER EUTROPHICATION FROM LAND DISPOSAL OF SEWAGE

M.J.J. Dayton and A. Berzins
Consulting Engineers, Dayton & Knight Ltd.

With increased awareness of water pollution, land disposal of sewage in the form of agricultural irrigation is becoming an attractive method for water reuse.

The practice is particularly favoured in areas where surface water eutrophication caused by phosphorus discharge is of concern. This is because the fertilizing capability of phosphorus for aquatic eutrophication is diverted towards fertilization of land crops and a multifold benefit is apparently derived.

While in some cases such water reuse is indeed an excellent practice resulting in several benefits in the form of elimination of surface water pollution, eutrophication control, enhanced agricultural production and abundant irrigation water, there are instances where diversion of wastewater from streams to agricultural irrigation should be approached with caution.

The arid Okanagan Valley in British Columbia provides one such example.

Okanagan Valley and its Water Use and Demand

The Okanagan Valley in the British Columbia interior is located about 200 miles inland from the Pacific Ocean.

It is about 100 miles long and averages 30 miles in width. The basin drains southward into Washington State and is eventually tributary to the Columbia River.

Its general location is shown in Figure No. 1.

The Okanagan Valley is in the Canadian portion of the North American Cordillera High Plateau, which experiences summer temperatures in excess of 100°F and a yearly precipitation of less than 10-inches.

As a result the valley supports an extensive irrigated orchard industry which started in the last decade of the previous century and reached almost the extent it occupies today by 1920. Today the orchards comprise some 60,000 acres of land which use about 100,000 acre feet of water for irrigation per year.

A second important feature in the Okanagan Basin is provided by an extensive lake chain with a theoretical average water flow

through rate in the neighbourhood of 50 years. The lakes are shown in Figure No. 2.

The lakes provide an attraction to both permanent settlement and to summer tourists. This has resulted in a permanent population growth from 20,000 in 1921 to 150,000 in 1976. The seasonal visitors, however, are now estimated to exceed 750,000.

Projections for the future indicate that the permanent population could reach 430,000 and the number of visitors 2,300,000 by the year 2020.

The permanent population, the non-agricultural industry, and the tourist population are all projected to use water at a rate increasing from 15,000 acre feet per year in 1976 to about 100,000 acre feet per year in 40 years time. If that trend were to continue, nearly 300,000 acre feet of water would be used by municipalities and industry in the long term future.

In addition to the irrigated orchards, the people, and industry, the basin contains another 100,000 acres of excellent agricultural land which is currently not used nor irrigated but which is projected to be put to agricultural usage in the foreseeable future. Its potential water demand is in the order of 150,000 acre feet per year.

Finally, there are about 300,000 acres of unforested range land, some of which has recently been converted to agricultural production using treated municipal wastewater. There are plans to turn additional range land into agricultural production using municipal sewage for irrigation.

In summary, the water use in the Okanagan watershed has increased, and can be projected to increase further in the future, as follows and as illustrated in Figure No. 3:

Yearly Water Use in Acre-Feet

	<u>1880</u>	<u>1930</u>	<u>Today</u>	<u>2020</u>	<u>Long Term Future</u>
Irrigated Orchards	Nil	100,000	100,000	100,000	100,000
Residential- Industrial	Nil	Little	15,000	100,000	300,000
Potential Agricultural Irrigation	Nil	Nil	Little	150,000	150,000
Rangeland Irrigation (by Resid.-Ind. Wastes)	Nil	Nil	Little	100,000	300,000
	<u>-</u>	<u>100,000</u>	<u>115,000</u>	<u>450,000</u>	<u>850,000</u>

Outflow from the Valley is about 400,000 acre feet per year from an average precipitation of nearly 4,000,000 acre feet per year over the past sixty years.

Water reuse is inevitable if the projected future water demand is to be met.

Apparent Phosphorus Loadings to the Valley Lakes

The growth in the permanent population, as well as in the number of tourists, has focused public attention on water quality in the large lakes.

There are few reliable long term data on lake water quality available for this basin. Spectacular algae blooms in the 1960's and an Eurasial Millfoil invasion in the 1970's, resulted firstly in extensive studies of the lake water quality and secondly in demands for better pollution control.

A joint Canada-British Columbia Comprehensive Study was carried out on the basin between 1972 and 1974 at a cost of over two million dollars. The main conclusion of that study centered around the

calculation that there was enough water available in the system to satisfy the 40 year projected water demands as long as the lake system was drawdown during drought conditions and during the peak summer usage interval. The study skimmed over the effects on water quality from such drawdowns, though a wealth of data was collected in the course of completing that study.

It was established that the phosphorus was the main element causing the increase in the aquatic productivity and that its content in some of the smaller downstream lakes such as Skaha and Osoyoos exceeded 0.1mg/l, and at certain times exceeded 0.05 mg/l in the major Okanagan Lake itself. All lakes were concluded to be fundamentally eutrophic or nearly so. From the studies it also became apparent that a major contributor of phosphorus loading was the discharges of sewage effluent from the major population centres in the Valley. It was estimated that the point sources of wastewater discharge at that time contributed 100,000 lbs. of phosphorus per year while the other non-point sources added an additional 300,000 lbs. per year. It was also established that the non point source contribution of phosphorus is related to agricultural land use.

The demands for improved pollution control have been based on the high phosphorus content in the lakes, and have resulted in one of the three major population centres, the City of Vernon, completely removing its treated effluent discharge from the lake and the two others contemplating such action for the near future.

The wastewater discharge thus removed from the lakes is in turn used to convert some of the range land into agricultural production to grow forage crops.

In the "phosphorus addition to the lakes" balance sheet, a complete removal of phosphorus by the elimination of wastewater point source discharges would then appear to take the following form. (also illustrated in Figure No. 4):

Apparent Contributions of Phosphorus In Lbs. per Year
To the Okanagan Valley Lakes

	Alternative A: No Wastewater Discharge Control				Alternative B: Complete Wastewater Discharge Elimination			
	1880	1930	Today	2020	1880	1930	Today	2020
Point Sources -		Little	100,000	1,500,000**	-	Little	100,000	Nil
Non Point Sources	100,000	200,000	300,000	300,000	100,000	200,000	300,000	300,000
	100,000	200,000	400,000	*1,800,000	100,000	200,000	400,000*	300,000

* Canada-British Columbia 1974 Comprehensive Study
 ** 100,000 acre feet or 75 mgd, containing 5.5 m/l phosphorus as P.

On the surface such water reuse would appear to reduce the contribution to the Valley system from 1,800,000 lbs. per year to 300,000 lbs. per year and seems to be an excellent approach towards a four fold goal:

- (1) Pollution Control in Surface Waters.
- (2) Eutrophication Control in Surface Waters.
- (3) Enhanced Agricultural Production.
- (4) Ample Irrigation Water.

The reuse of wastewater for agricultural irrigation has accordingly been encouraged and is now favoured by many organizations and people, both locally and on a Canada wide basis.

A More In-Depth Analyses of Water Quantity and of Phosphorus Balance in Valley Lakes

There is one aspect in the water reuse planning, however, which has largely been neglected. That aspect centres around a delicate water quantity balance in the years to come.

Accurate estimates of lake water throughflow for the Okanagan Basin are difficult to formulate because of highly variable evaporation evapotranspiration conditions in the watershed as well as normal variations in precipitation.

However, good records of the downstream watershed discharge are available for the last 60 years. Similarly investigations of

irrigation return flow to the groundwater regime have now been made both for irrigation of the orchards and for irrigation of forage crops.

These indicate that the mean lake discharge between 1915 and 1975 has been in the order of 350,000 acre feet per year and that the net irrigation return flow to the lake system, in conformity with all other well operated irrigation practices, varies between 10 and 25 percent of the applied water.

If new agricultural irrigation is initiated in the years to come that reuses municipal and industrial wastewater as proposed, the average yearly water balance in the Okanagan Valley Lakes will change as tabulated below and illustrated in Figure No. 5:

In Depth Analysis of Long Term Average Yearly Water Balance
in the Okanagan Basin System in Terms of Acre-Feet

	<u>1880</u>	<u>1930</u>	<u>Today</u>	<u>2020</u>	<u>Potential Long Term Future</u>
Long Term Average Discharge from Okanagan Valley System in absence of man made activities	440,000	440,000	440,000	440,000	440,000
Less Water Used and Not Returned to Nature's Cycle by:					
a) Orchard irrigation	Nil	90,000	90,000	90,000	90,000
b) Municipal/industrial usage and Rangeland Irrigation	Nil	Nil	5,000	90,000	270,000
c) Potential Agricul- tural Irrigation	Nil	Nil	Nil	135,000	135,000
Net Long Term Average Discharge from Okanagan Valley after man-made activities	440,000	350,000	345,000	125,000	Nil

The result of such reuse is then, of course, that while the apparent phosphorus contribution to the lakes is decreased from 1,800,000 to 300,000 lbs. per year, the theoretical flow through rate in

the Valley lakes is increased from a time frame of 50 years today to 140 years in the year 2020 and to infinity in the long term future.

Furthermore, if non point source contribution of phosphorus to the Valley lakes is indeed a function of agricultural acreage, the non point contribution of phosphorus will increase with the development of agricultural land as follows:

Projected Development of Agricultural Land
and Non Point Contribution of Phosphorus

	<u>1880</u>	<u>1930</u>	<u>Today</u>	<u>2020</u>	<u>Potential Long Term Future</u>
Agricultural Acreage	Nil	60,000	60,000	160,000	460,000
Non Point Contribution of Phosphorus (lbs./year)	100,000	200,000	300,000	-	-

This is illustrated in Figure No. 6.

The effects on pollution and eutrophication from the prolonged lake water flow-through rate and from potential additional non point source contribution of phosphorus should receive considerably more attention than what is now the case because both are the classic causes of the premature aging of lakes in addition to point source contributions of phosphorus from growing shoreline populations.

Some Difficult Questions on Eutrophication

In summary it has been projected that apparently:

- (A) The phosphorus contribution to the Okanagan Lake system can be decreased from 1,800,000 lbs. per year to 300,000 lbs. per year in 40 years time if all municipal/industrial wastewater was reused for irrigation.

As a result, this has become a favoured policy for pollution and eutrophication control in the Valley.

The other dimension to this policy has recently been voiced, however, and is starting to raise concern:

- (B) - The average lake system discharge will decrease from 345,000 acre feet per year to 125,000 acre feet per year in 40 years time and to zero in the long term future.

- The theoretical lake flow through time, accordingly, will increase from 50 years to nearly 140 years and eventually to infinity in the same time frame.
- The irrigation reuse will increase the non point source contribution of phosphorus.

The agricultural irrigation and the slow down in lake discharge would result in several changes:

- (1) The average lake water temperature will increase in the summer and decrease in the winter.
- (2) The lake evaporation rates will increase accordingly.
- (3) The release of phosphorus from bottom sediments with decreasing dissolved oxygen levels resulting from slower lake turn-overs will increase.
- (4) The non point return flow from irrigation will increase.
- (5) The discharge of phosphorus from the system by flushing will decrease.

While the effects on biological productivity from the first three of the above items is difficult to assess, the effects of the fourth and the fifth are more apparent.

Today over 100,000 lbs. of phosphorus are flushed from the lake system under average conditions. If the flow through time is increased to 145 years, the flushing of phosphorus decreases to 35,000 lbs. per year and stops with no positive discharge in the long term future.

If this is taken into consideration, the phosphorus balance sheet set out in Figure No. 4 changes slightly as tabulated below and illustrated in Figure No. 7.

Net Yearly Inflow of Phosphorus In Lbs. per Year
To Okanagan Valley System Allowing For Flushing

	<u>Alternative A.</u>		<u>Alternative B</u>	
	<u>No Wastewater Discharge Control</u>		<u>Complete Wastewater Discharge Elimination</u>	
	<u>Today</u>	<u>40 Year</u>	<u>Today</u>	<u>40 Year</u>
Point Sources	100,000	1,500,000	100,000	Nil
Non Point Source	300,000	300,000	300,000	300,000
	400,000	1,800,000	400,000	300,000
Less Flushing	100,000	100,000	100,000	35,000
	300,000	1,700,000	300,000	265,000

The effects of complete wastewater removal on the phosphorus balance still appear significant.

However, if in addition non point source contribution of phosphorus is related to agricultural land development under the assumption that non point source phosphorus is directly related to the agricultural acreage being irrigated as suggested by past studies, the Figure No. 4 balance could change dramatically as tabulated below and as illustrated in Figure No. 8:

Net Yearly Addition of Phosphorus in Lbs. Per Year
To The Okanagan Valley Lakes Allowing For Flushing
And Increased Non Point Source Contritutions
As a Result Of Increasing Agricultural Land Use

	<u>Alternative A</u>		<u>Alternative B</u>		
	<u>No Wastewater Discharge Control</u>		<u>Complete Wastewater Discharge Elimination</u>		
	<u>Today</u>	<u>40 Years</u>	<u>Today</u>	<u>40 Years</u>	<u>Long Term</u>
Point Sources	100,000	1,500,000	100,000	Nil	Nil
Non Point Sources	300,000	300,000	300,000	800,000	2,300,000
	400,000	1,800,000	400,000	800,000	2,300,000
Less Flushing	100,000	100,000	100,000	35,000	Nil
	300,000	1,700,000	300,000	765,000	2,300,000

Instead of the apparent decrease in phosphorus contributions to the lake system from 400,000 to 300,000 lbs. per year, complete reuse of wastewater for agricultural irrigation is liable to increase the phosphorus contributions to over 2,000,000 lbs. per year in the long term if some of these less apparent criteria are taken into consideration.

In arid areas, intensified agricultural production may well be the single most important contributor to surface water pollution considerably overshadowing the more apparent and more tangible contribution from growth in population and in industrial development.

Conclusion

In a water short area such as the Okanagan Valley in British Columbia, wastewater reuse for agricultural irrigation should be approached with caution, and should only be implemented after a study of the potential long term effects on water quality and quantity.

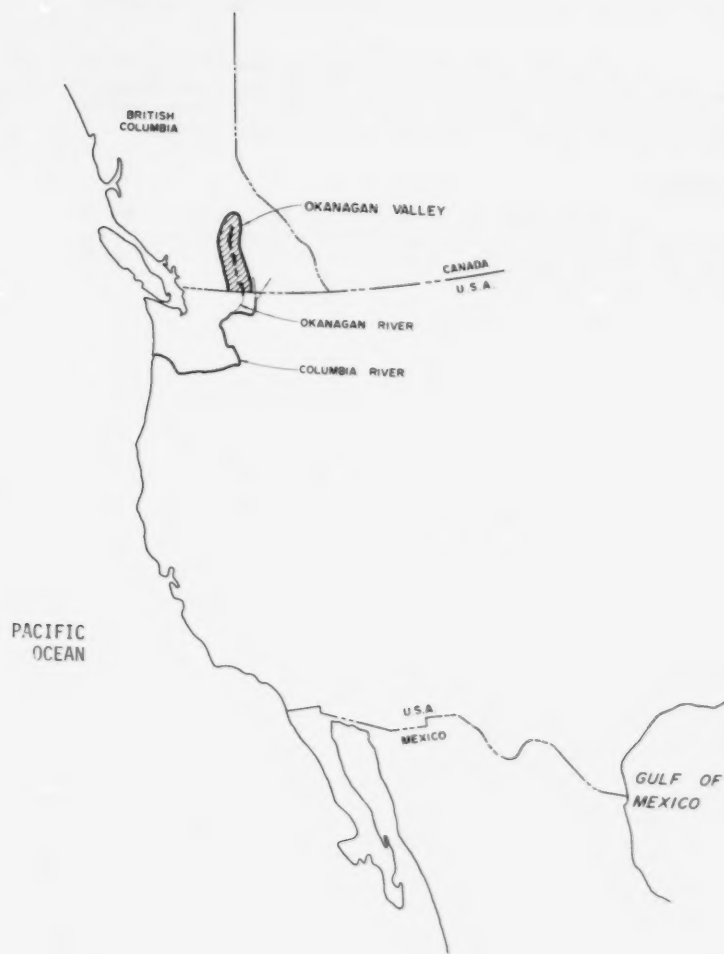


FIGURE 1

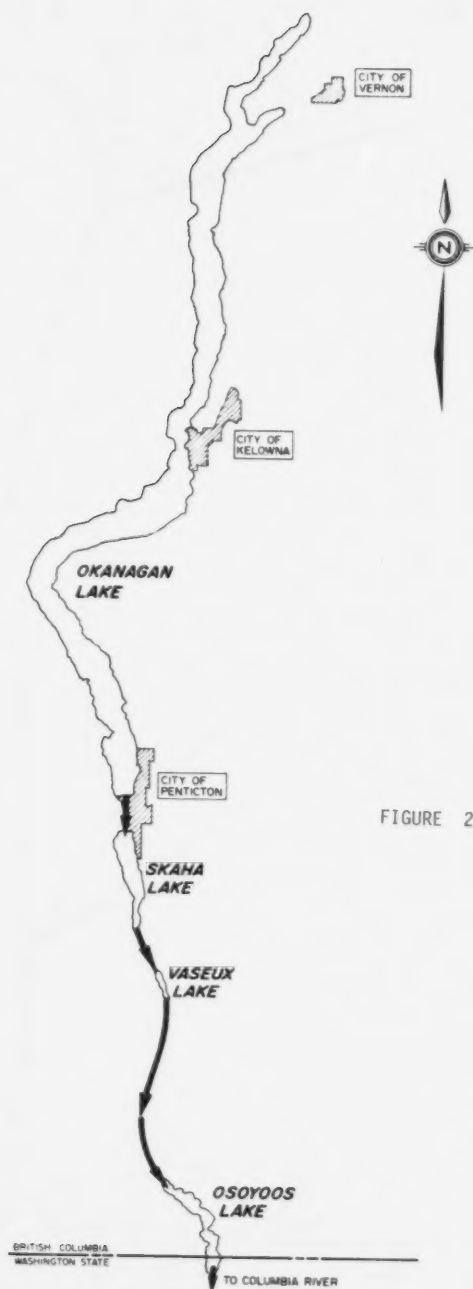
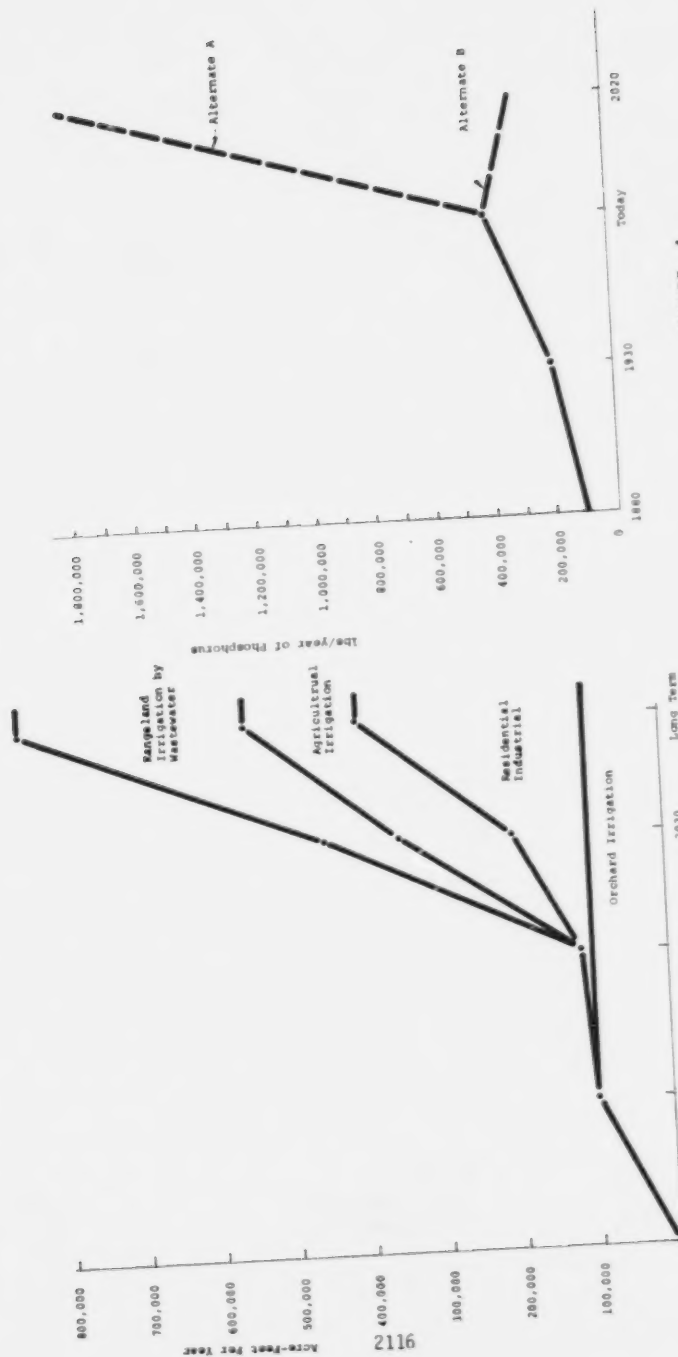


FIGURE 2



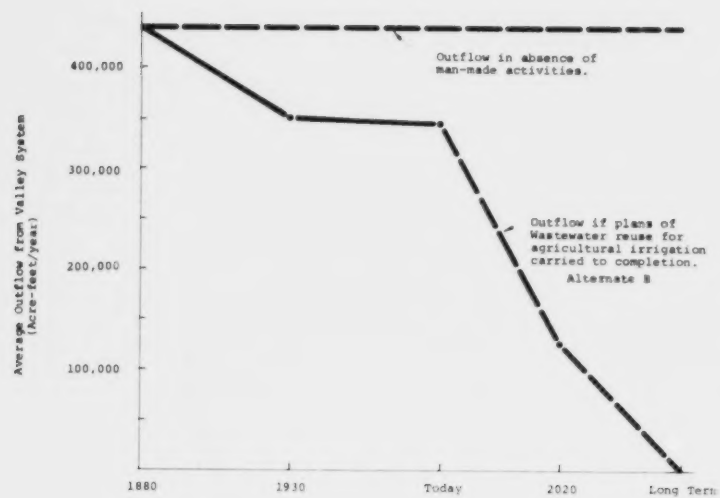


FIGURE 5

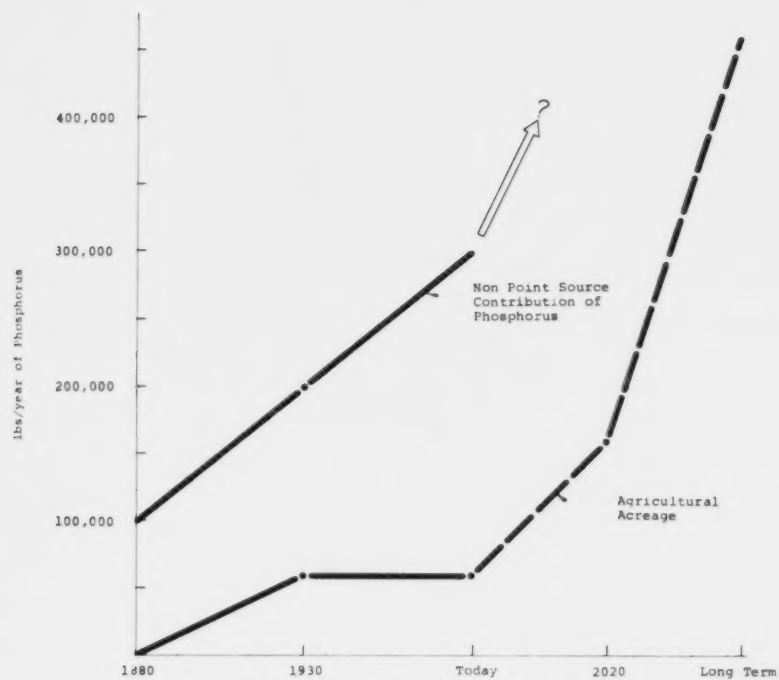
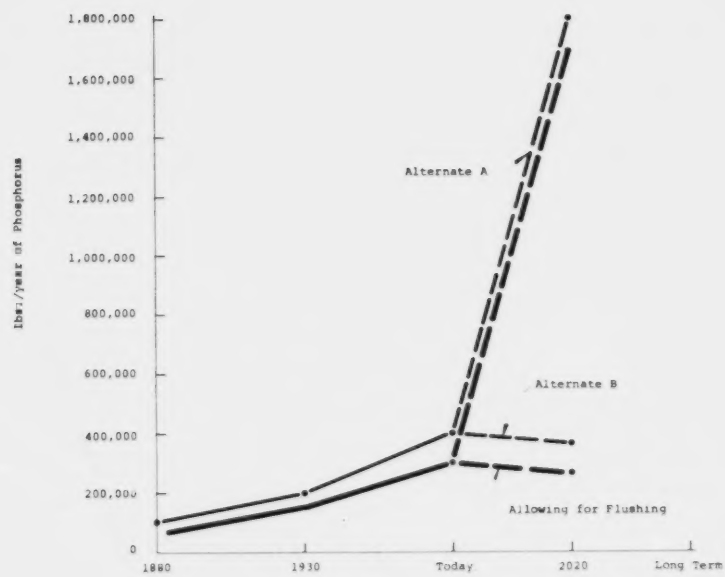
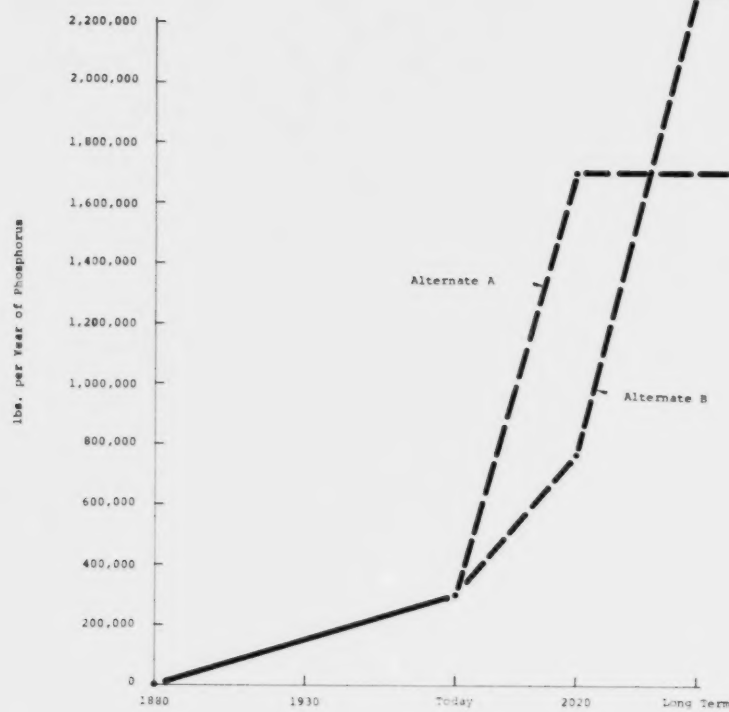


FIGURE 6



PHOSPHORUS CONTRIBUTION
INCLUDING FLUSHING

FIGURE NO. 7



PHOSPHORUS CONTRIBUTION
INCLUDING FLUSHING AND INCREASING
AGRICULTURAL LAND USE

FIGURE 40. 3



HEALTH AND QUALITY CONSIDERATIONS

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WASTEWATER RECLAMATION IN CALIFORNIA

James Crook
Senior Sanitary Engineer
California Department of Health Services

David P. Spath
Senior Sanitary Engineer
California Department of Health Services

INTRODUCTION

The State of California recognizes that treated domestic wastewater is a valuable resource and that reclaimed water will play an ever-increasing role in helping to meet the water demands of the state. The State Legislature declared, in 1969, that a substantial portion of the future water requirements of the state could be economically met by beneficial reuse of water (1). California's interest in reuse was reconfirmed in 1977 by establishment of an Office of Water Recycling to coordinate all of the state's reclamation activities. For many years, the Department of Health Services has encouraged wastewater reuse where public health protection is not compromised, thus conserving the present domestic water sources for higher-order uses.

In general, the responsibility of the Department of Health Services in the area of wastewater reclamation is to assure public health protection from pathogenic agents and toxic substances. This is accomplished by several means, including surveillance, regulations, guidelines, policies, consultation with other agencies, evaluation of treatment effectiveness and other research, and dissemination of information. The Department of Health Services works closely with project proponents, consulting engineers, local health departments, and other state and Federal agencies to ensure that all health-related issues are addressed for any proposed or existing use of reclaimed water.

The state-of-the-art of wastewater treatment and treatment reliability is at a point where sewage effluent can be used for a multitude of purposes when reasonable precautions are taken in its use. At the present time, California has approximately 220 wastewater reclamation plants supplying water to more than 350 discrete use areas. The types of reuse and accompanying numbers of use areas are shown in Table 1. It should be noted that none of the eight food crop irrigation operations involved direct contact between the reclaimed water used for irrigation and the edible portion of any unprocessed food crop.

In 1978 approximately 184,000 acre-feet ($230 \times 10^6 \text{ m}^3$) of wastewater was reclaimed in the state by planned reclamation operations, which is about 7 percent of the total wastewater produced. As indicated in Table 1, over 60 percent of the wastewater reused in California is for uses that require a minimal degree of treatment, i.e., primary settling. These uses include fodder, fiber, or seed crop irrigation, and the surface irrigation of orchards or vineyards. Irrigation of all kinds accounts for almost 80 percent of the total quantity of wastewater reclaimed in the state. It has been estimated that as much as 1,700,000 acre-feet/yr ($2,100 \times 10^6 \text{ m}^3/\text{yr}$) is available for reclamation (2), which is equal to 5 percent of the present total water demand in California.

HEALTH ASSESSMENT

Clearly, most wastewater reclamation and reuse operations impose a greater risk of public or worker exposure to pathogenic organisms or toxic substances than would occur if waters of non-sewage origin were used. The objective, therefore, is to minimize the risks and reduce the potential health hazards to acceptable levels. In general, the health concern is in proportion to the degree of human contact with the water and the adequacy and reliability of the treatment processes. The health risks may be minimal where exposure to the wastewater by contact, ingestion, or inhalation is unlikely, but the risks increase when direct or indirect exposure is probable.

The contaminants in reclaimed water that are of health significance may be grossly classified as biological and chemical agents. Historically, the biological agents have been the ones receiving the closest attention and, for most of the uses of reclaimed wastewater, quality standards are properly directed at these agents. Higher uses of reclaimed water, where the public is more directly exposed and ingestion of the reclaimed water is more likely, will also require control of chemical contaminants.

Some of the potential waterborne diseases include bacillary and amoebic dysentery, salmonellosis, leptospirosis, cholera, vibriosis, and infectious hepatitis. Other less common diseases are brucellosis, ascariasis, schistosomiasis, ancylostomiasis, and tapeworm disease. Epidemiological evidence from other countries indicates that reuse of sewage, particularly for crop irrigation, has resulted in several disease outbreaks (3); however, the reuse of treated wastewater in California has not resulted in any confirmed disease outbreaks.

Sanitary engineering and preventive medicine have combined to reach a point where waterborne disease outbreaks of epidemic proportions have, to a great extent, been controlled; however, the potential for disease transmission through the water route has not been eliminated. With a few exceptions, the disease organisms of epidemic history are still present in today's sewage, and the status of control is more one of severance of the transmission chain than a total eradication of the disease agent. As an example, vegetable decontamination studies have indicated that washing vegetables with plain water or detergents is ineffective in removing bacteria and helminth eggs. Helminth eggs are also extremely resistant to chemical disinfectants. Consequently, in the case of food crops, emphasis should be placed on either eliminating the pathogenic agents from the wastewater prior to irrigation or preventing direct contact between the wastewater and the edible portion of the crop to minimize the risks of disease transmission.

Increasing attention has been given to the transmission of viral diseases through the water route. Waterborne outbreaks of infectious hepatitis have clearly been associated with the ingestion of sewage contaminated waters (4,5). Identification and enumeration of viruses in water and wastewater has been hampered by the limitations of sampling techniques, problems of concentration of samples, the complexity and high cost of laboratory procedures, and the limited number of facilities having the personnel and equipment necessary to perform the analytical service.

There is ample evidence that many viruses are more resistant than bacteria to the usual disinfection practices (6,7,8). This, coupled with the many problems of virus monitoring, makes assurance of virus

destruction or removal a difficult matter in wastewater treatment. In this regard, the Department of Health Services is attempting to gather data on the presence of virus in treated domestic wastewater. A water virology laboratory has been established within the Viral and Rickettsial Laboratory and works closely with the Department's Sanitary Engineering Section. One of the laboratory's first tasks will be a study to evaluate the viral content of reclaimed waters that are used for various types of landscape irrigation.

The possibility of disease transmission by aerosols from spray irrigation sites must also be considered where the source of water is sewage effluent. The potential impact of aerosols is becoming a major issue, due to the proliferation of reuse projects in urban settings or adjacent to populated areas. The degree of hazard depends on several factors, including degree of wastewater treatment, extent of aerosol travel, proximity to populated areas or areas accessible to the public, prevailing climatic conditions, and design of the irrigation system. In general, bacteria and viruses in aerosols remain viable and travel farther with increased wind velocity, increased relative humidity, lower temperature, and darkness (9,10). Airborne pathogenic organisms may remain viable for extended time periods; however, effective wastewater treatment and disinfection in conjunction with proper use area controls, can reduce the health risks to acceptable levels.

It should be recognized that the acceptability of using wastewater for crop irrigation is dependent upon its chemical quality as well as its biological quality. Some chemical constituents accumulate in particular crops, thus presenting health hazards to both grazing animals and/or humans. For example, once in the soil system, some plants such as clovers and grasses will take up heavy metals to concentrations that are toxic to the consumer but not to the plant. There is a reasonably clear understanding of crop uptake and the acute health significance of heavy metals and the more toxic chemical agents (11,12) and these materials are usually reduced to acceptable levels in domestic wastewater by conventional sewage treatment processes (13).

Direct or indirect reuse of wastewater for domestic purposes encompasses a much broader range of public health risks than lower-order uses. For uses involving ingestion, health effects due to prolonged exposure to low levels of organic and inorganic chemical contaminants must be considered as well as the acute health effects from pathogens or toxic substances.

The chemical agents of health concern that may be present in reclaimed water include a wide variety of organic and inorganic chemicals, such as heavy metals, pesticides, and chlorinated hydrocarbons. Although the mineral composition of a reclaimed water entering a domestic water supply by any route may have an impact on public health, it is not considered to be different in character from the health effect of the same mineral constituents occurring naturally. Generally, there are established limits for such mineral constituents and, although reclaimed waters are more mineralized than their source waters and more likely to contain heavy metals or nitrates than most natural waters, such materials can be easily detected and removed or modified by treatment to prevent a health hazard.

Some information is available regarding the chronic effects from ingestion of trace amounts of metals, and permissible levels of these constituents have been established for drinking water. Information on the presence and concentration of organic constituents, however, is

extremely scarce and there is increasing concern that low concentrations of these substances may produce long-term health effects, i.e., carcinogenesis, mutagenesis, and teratogenesis.

Indirect reuse of wastewater through the process of incidental groundwater basin recharge with sewage effluents has occurred for many years, generally constituting a minor fraction of the input into the basins. Natural soil systems are effective for improving water quality, including the removal of certain soluble organics as well as suspended materials, bacteria, and viruses. When soil types, depths, and geological conditions are suitable, such systems are reliable and not subject to sudden failures.

The health-related issues associated with direct potable reuse are much more difficult to resolve, particularly in relation to trace organic substances. Before any agency can proceed in a responsible way to plan for direct reuse for potable purposes, standards must be set for organic compounds. This, in turn, requires knowledge of the constituents likely to be present, information on the toxicology of the substances, and development of engineering data on the removal of such substances by conventional or advanced treatment processes. It is obvious that a great deal of research has yet to be conducted, and projects involving direct potable reuse of wastewater should be deferred until a reasonable amount of scientific evidence is available. In the interim, water reuse projects can and should proceed for end uses that do not require water of potable quality.

WASTEWATER RECLAMATION CRITERIA

The Department of Health Services has been given the authority and responsibility to establish wastewater reclamation criteria and has done so for uses involving irrigation, impoundments, and groundwater recharge. However, the Department does not have any direct enforcement powers regarding these regulations. The Regional Water Quality Control Boards have the authority to set and enforce reclamation standards and requirements. Each regional board, after consulting with and receiving the recommendations of the State Department of Health Services and local health departments, prescribes water reclamation requirements as part of the waste discharge permit. Requirements may be placed upon the person reclaiming the water, the user, or both. If, however, a wastewater reclamation project causes a condition of contamination to exist, the Department of Health Services has the authority to issue a peremptory order requiring abatement of the contamination.

California has had regulations governing the use of sewage for irrigation purposes for over 60 years. A basic objective of the Wastewater Reclamation Criteria (14), most recently revised in 1978, is to assure health protection without unnecessarily discouraging wastewater reclamation.

The regulations establish acceptable levels of constituents of reclaimed water and prescribe means for assurance of reliability in the production of reclaimed water to ensure that reuse for the specified purposes does not impose undue risks to health. Water reclamation is in the best public interest and the declared policy of the State is to encourage reclamation. The reclamation criteria are intended to promote development of facilities which will assist in meeting water requirements of the State while assuring positive health protection. Of course, in addition to compliance with the Wastewater Reclamation

Criteria, appropriate surveillance and control of treatment facilities, distribution systems, and use areas must be provided in order to avoid health hazards.

Water Quality

The wastewater treatment and quality requirements increase as the health hazards associated with the reuse of the wastewater increase, as shown in Figure 1. For example, the irrigation of fodder, fiber, or seed crops presents a rather low-order health risk (assuming proper application and use area controls), and primary effluent containing not more than 0.5 ml/l/hr of settleable solids is acceptable. However, the possibility of disease transmission is much greater if food crops are spray irrigated with treated wastewater; therefore, the regulations for this use are more restrictive and require that the wastewater receive secondary treatment followed by chemical coagulation, clarification, filtration, and disinfection, and have an effluent median MPN limit of 2.2 total coliform organisms per 100 ml. It should be pointed out that methods of treatment other than those specified in the Wastewater Reclamation Criteria and their reliability features may be acceptable if they are demonstrated to assure an equal degree of treatment and reliability.

For most uses of reclaimed water the regulations do not require an extensive monitoring program to demonstrate reclaimed water quality, because it would eliminate the many small reclamation operations which would not be able to afford the expense of a sizable monitoring load. Consequently, insofar as possible without jeopardizing the intent of the standards to provide health protection, descriptive terms which are well-understood in the wastewater treatment field are used rather than limits of specific parameters. For example, an "adequately oxidized wastewater" is required rather than effluent meeting a specific BOD, suspended solids, or other parameter.

There has been increasing interest in California by planners, engineers, some water management entities, and others to have the Department of Health Services develop criteria for reclaimed water used to replenish groundwaters which are sources of domestic water supply. Because of the lack of information on long-term health effects from prolonged domestic use of groundwaters containing varying amounts of treated wastewater, and in the absence of recognized standards elsewhere, the Department of Health Services has found it extremely difficult to develop criteria for groundwater recharge.

In 1975, the State Water Resources Control Board, Department of Water Resources, and the Department of Health Services established a "Consulting Panel of the Health Aspects of Wastewater Reclamation for Groundwater Recharge." This group, composed of nationally known experts in the areas of water supply, wastewater treatment, groundwater hydrology, toxicology, epidemiology, risk-benefit analysis, and consumer affairs, was asked to study the problems involved with groundwater recharge with reclaimed water and recommend and research a program to provide information that could lead to the development of appropriate criteria. In their report, the Panel pointed out that stable organic materials are the principal concern associated with groundwater recharge with reclaimed water. The Panel also concluded that, because there have been no apparent adverse health effects (studies of health effects have not been conducted) associated with properly operating planned groundwater recharge operations, existing projects should continue in operation. However, since monitoring and

surveillance have not been directed at the health aspects, comprehensive studies are urgently needed. In addition, it was recognized that, due to the uncertainties concerning health effects, it may be necessary to set criteria on new projects so restrictive as to make groundwater recharge economically unattractive.

The Department, with the guidance of an informal consulting group, drafted regulations and held public meetings to present the draft regulations and receive input and comments on them. Based on the input received, the draft regulations were revised and additional workshop-type meetings were held with various agencies and organizations to further discuss the draft regulations. It became apparent during the meetings that, due to the uniqueness of each individual groundwater basin, it was exceedingly difficult, if not impracticable, to adopt detailed statewide standards for groundwater recharge by surface spreading.

Ultimately, it was decided that groundwater recharge projects should be evaluated on an individual case basis. Therefore, the Wastewater Reclamation Criteria were revised to contain a section which states, in part, that "Reclaimed water used for groundwater recharge of domestic water supply aquifers by surface spreading shall be at all times of a quality that fully protects public health. The State Department of Health Services' recommendations to the Regional Water Quality Control Boards for proposed groundwater recharge projects and for expansion of existing projects will be made on an individual case basis where the use of reclaimed water involves a potential risk to public health."

The previously developed draft regulations will be used as a basis for general guidelines to be developed for the case-by-case evaluation. This procedure has several advantages. Each proposal can now be evaluated on its own merits considering wastewater volumes, treatment, reliability, operating practices, geology, hydrological conditions, and other features. Also, the guidelines can be continually updated as new information becomes available, particularly in the areas of treatment technology, monitoring, and the identification and health significance of trace organic constituents. The basic guidelines will include various measures of control to assure a reasonable degree of public health protection, including but not limited to: wastewater treatment facilities, performance, and reliability; wastewater quality, quantity, and monitoring; operational practices; required studies and reports; and evaluation of soil characteristics, hydrogeology, residence time, and distance to withdrawal.

At the present time, regulations have not been developed for uses such as fire protection, soil compaction, dust control, industrial reuse, or landscape irrigation of residential areas. Proposals to reuse wastewater for these or other purposes not covered by the regulations are evaluated on an individual case basis.

Treatment Reliability

The need for adequate treatment is obvious, but it is not so clearly recognized that there is an equally important need to assure reliability of treatment. The increase in reclamation operations and the more frequent use of reclaimed wastewater in public areas has increased the population which may be exposed to wastewater; consequently, the potential for illness resulting from an improperly treated water being delivered to the use areas has also increased. Thus, it is apparent

that provisions are necessary to insure reliability of treatment if a minimum health risk is to be maintained during the reuse of treated sewage effluent.

The Wastewater Reclamation Criteria contain both design and operational requirements necessary to ensure an appropriate level of treatment reliability. The regulations require essential reliability features such as alarm systems, standby power supplies, treatment process reliability (multiple or standby units, replacement parts, etc.), emergency storage or disposal of inadequately treated wastewater, elimination of treatment process bypassing, monitoring devices and automatic controllers, and flexibility of design.

From a public health standpoint, provisions for adequate and reliable disinfection are the most essential features of the treatment process. Where disinfection is required, the reclamation criteria specify that the following features must be incorporated into the system to insure uninterrupted chlorine feed: standby chlorine supply; manifold systems to connect chlorine cylinders; chlorine weighing scales; and automatic devices for switching to full chlorine cylinders. As with other unit processes, several alternatives are available to meet the treatment process reliability requirements. As an example, the regulations state that all disinfection unit processes shall be provided with one of the following reliability features: (a) alarm and standby chlorinator; (b) alarm, short-term retention or disposal provisions, and standby replacement equipment; (c) alarm and long-term storage or disposal provisions; (d) Automatically actuated long-term storage or disposal provisions; or (e) alarm and multiple point chlorination, each with independent power source, separate chlorinator, and separate chlorine supply.

Several field investigations of municipal sewage treatment plants in California have documented that wastewater treatment reliability has been a neglected phase of treatment design and development (15). A 1964 study of 312 treatment plants indicated that 56 percent of the plants experienced equipment outages during the preceding year. Thirty-three percent of the plants reported the necessity to bypass untreated sewage for periods ranging from 6 days to 10 months. A 1969 study of 45 reclamation facilities revealed that approximately 40 percent of the plants did not have an acceptable form of emergency disposal provisions, and 70 percent of the plants did not have any type of emergency power source. Only 3 of the 45 plants were conducting daily bacteriological examinations on the reclaimed water.

A 1975 study (16) of 194 wastewater reclamation facilities in California determined that standby power supplies were provided at less than half of the reclamation plants and only 25 percent were equipped with adequate alarm systems. Further, over 60 percent of the facilities producing wastewater for uses requiring a disinfected effluent did not provide acceptable disinfection reliability. The need for reliability was accentuated by the finding that only 14 percent of the treatment plants were manned by qualified personnel on a continuous basis.

The Department of Health Services conducted a comprehensive study of 220 wastewater reclamation treatment plants in 1978. Data from that study indicate that adequate treatment reliability is still lacking at a majority of the facilities. For example, only 22 percent of the facilities had adequate alarm systems and 39 percent were provided with standby power supplies. Two-thirds of the reclamation facilities provided for emergency storage or disposal of inadequately treated wastewater as required by the Wastewater Reclamation Criteria, but only

47 percent had an acceptable level of unit process treatment reliability, exclusive of disinfection. In the opinion of the staff who conducted the field study, less than 10 percent of the treatment plants that were required to disinfect the effluent provided adequate disinfection reliability. Of the 74 facilities that had bacteriological limits placed on their effluents, only 61 percent complied with the requirement that coliform samples be collected and analyzed daily, whereas 47 percent discharged effluent that did not meet the bacteriological requirement at least once during the year preceding the study.

Use Area Controls

It was previously stated that the regulations for any specific use are based on the expected degree of contact with the reclaimed water. The anticipated degree of contact, in turn, is based on compliance with proper design and operational controls at the use area. In recognition of the need to minimize health risks at the point of reuse, the Department of Health Services has developed use area guidelines that describe appropriate safety precautions and operational procedures, e.g., cross-connection control provisions, color-coded reclaimed water lines and appurtenances, key-operated valves and outlets, fencing, signs, control of aerosols, and provisions for worker protection. Information concerning wastewater conveyance facilities is included in the use area guidelines, e.g., recommended separation and construction criteria for domestic and reclaimed water transmission and distribution pipelines.

SUMMARY

The State of California has been a leader in the area of wastewater reclamation and, in conformance with the official State position to encourage reclamation, will continue to actively promote reuse for uses that do not jeopardize public health. As part of a coordinated effort, the Department of Health Services has developed a comprehensive wastewater reclamation program in order to assure that the public is not exposed to unreasonable health risks. The health risks associated with many types of reuse are acceptable when adequate reclaimed water quality, treatment reliability, monitoring, and use area controls are maintained. In California, reclaimed water quality standards and treatment reliability requirements have been developed for uses involving irrigation, impoundments, and groundwater recharge. Public health authorities agree that direct reuse of wastewater for potable reuse should be deferred until enough scientific evidence is available to adequately assess the potential adverse health effects.

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TABLE I
TYPE AND QUANTITY OF WASTEWATER REUSE

Type of Reuse	Number of Use Areas	Quantity of Wastewater Reused (AF/Yr)	Percent of Total Quantity Reused (%)
Fodder, Fiber, and Seed Crop Irrigation	190	104,200	56.8
Landscape Irrigation - Golf Courses, Cemeteries, etc.	77	21,150	11.5
- Parks, Playgrounds, etc.	27	2,735	1.5
Orchard and Vineyard Irrigation	21	8,050	4.4
Construction and Dust Control	12	190	0.1
Industrial Uses	8	8,610	4.7
Food Crop Irrigation	8	4,970	2.7
Restricted Recreational Impoundments	6	2,300	1.3
Landscape Impoundments	6	2,135	1.2
Groundwater Recharge	5	25,950	14.2
Nonrestricted Recreational Impoundments	1	2,450	1.3
Wetlands - Marsh Enhancement	1	622	0.3
Aquaculture - Salmon - Rearing Ponds	1	153	<<0.1
TOTAL	363	~ 184,000	100

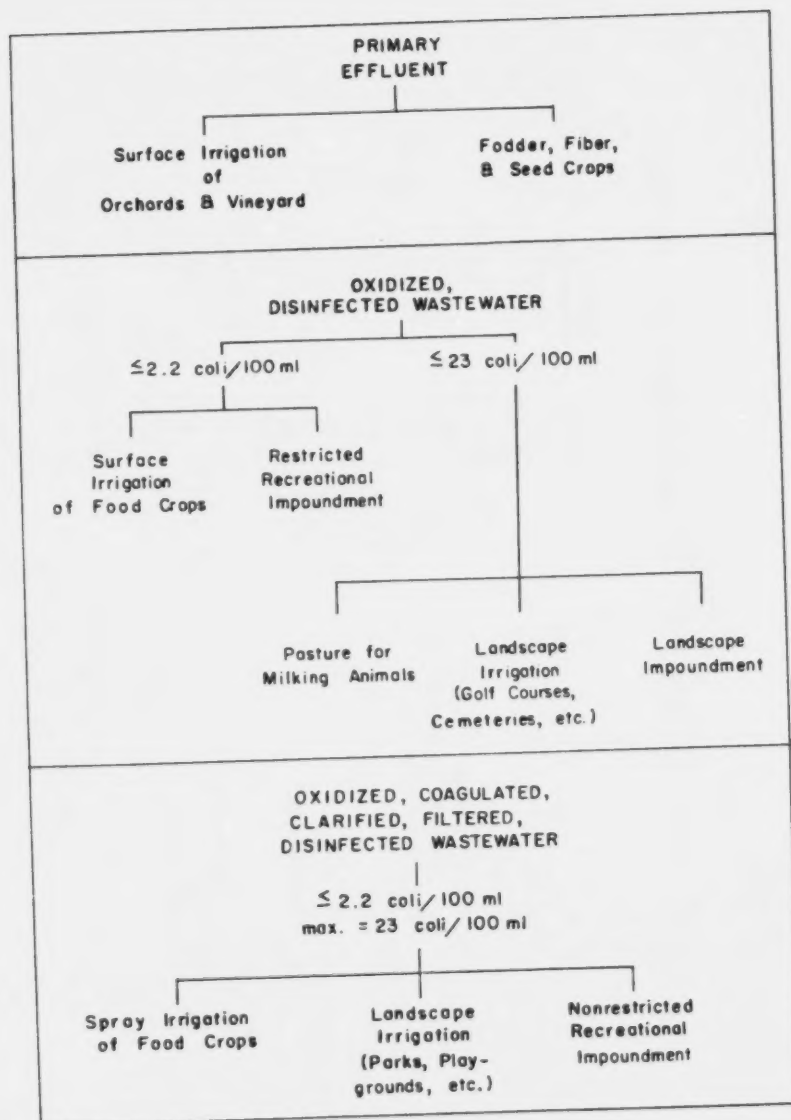


FIG. 1 WASTEWATER RECLAMATION QUALITY REQUIREMENTS

THE DETECTION OF HUMAN CELLULAR TOXICITY DUE TO ENVIRONMENTAL SUBSTANCES

Clive C. Solomons, Ph.D.
Research Professor, University of Colorado Medical Center

INTRODUCTION

To determine the total health effects of a substance in the environment is a formidable task. Because of the complexity of mammalian cellular metabolism and the large variety of thousands of interacting chemical reactions in the body, it is impractical to test a suspected toxin against every known metabolic step or sequence of steps. Even if this could be done, the ability of the cell to compensate, within limits, for many types of stress would make interpretation and extrapolation extremely difficult. The situation is simplified if the compound being evaluated has a high specificity for a single enzyme system. For example, organo phosphates (Parathion) specifically inhibit acetylcholinesterases and quickly disrupt the body's neuro-muscular system. However, apart from acute toxicity of this type, which is relatively easy to detect, we are more concerned about the effects of chronic exposure to much less acutely toxic substances of low concentrations. Therefore, a variety of compounds, including aromatics, paraffins, and their chlorinated derivatives, were studied, as well as fractions of unknown compositions derived from treated sewage and purified water. We also wished to measure the relative effectiveness of water purification steps such as charcoal treatment and reverse osmosis with a view to developing on-line biological testing as described in a new grant proposal.

The main difficulties of obtaining meaningful data can be categorized as follows:

- 1) Concentrations of test substances are low (from 1.0 to less than 0.1 ppm).
- 2) Exposure time for rapidly metabolizing cells *in vitro* is limited to 30-300 minutes to avoid the effects of normal deterioration of cellular metabolism.
- 3) Of all the possible pathways, which are the ones to be selected as the most valid as test models?
- 4) If cellular toxicity is detected *in vitro*, how does one extrapolate this data to evaluate gross human toxicity?

Nevertheless, an understanding of the short-term effects of chemicals on metabolic reactions had led to a sensitive detection system which reacts to a large number of compounds at a very basic level of cell metabolism.

CHOICE OF A METABOLIC PATHWAY

In attempting to deal with the problems of low concentration and relatively short periods of exposure, it is essential to use cells that are extremely sensitive to a very small concentration changes in their immediate environment, on the order of 10^{-6} - 10^{-5} M, and which react explosively and uncontrollably once they are triggered. Two types of cells, platelets and leucocytes, which normally

fulfill these requirements in the body, were studied.

Normally, when a blood vessel is cut or damaged the platelets aggregate to each other to form a plug which stops bleeding and acts as a catalytic surface for further coagulation. In the course of aggregation the platelet becomes degranulated, releasing a number of substances, including ADP, serotonin and a variety of proteins. Platelet function is linked closely to adenylate metabolism and is regulated by membrane function. Consequently, lipid soluble chemicals which change membrane function can affect platelet metabolism and behavior. This concept is important not only from the viewpoint of hemostasis, but is also related to the role of platelets in interacting with bacteria and viruses.

Another type of cell, which also functions explosively to a stimulus, is the white cell in the blood which forms one of the defenses against infection and malignancy. These cells can ingest or phagocytose bacteria and kill them with hydrogen peroxide generated metabolic action. Thus, substances which impair these functions or cause these cells to inappropriately attack the tissues of the host are of concern. The white cells, like the platelets, generate and use large quantities of high-energy-transducing purines (ATP, etc) and have efficient pathways for salvaging ATP from degradation products such as hypoxanthine (HYPX).

The third difficulty of choosing a relevant pathway to study is not as easily solved and all choices of metabolic pathways are open to some sort of criticism. However, there is one sequence which is of central importance to both the energizing and kinetic control of all metabolic steps. This system is the adenylate control system which has come to be recognized as being of prime importance in energy transduction, storage and control of metabolic rates and directionality. The importance of the fast-acting adenylate control system cannot be exaggerated for a large number of chemical reaction sequences which typify living organisms. The adenine nucleotides interact with all sequences in a complex living cell and uncontrolled changes in the relative concentrations of ATP, ADP and AMP would adversely affect the rates of all metabolic reactions and thus be highly disruptive.

The basis of adenylate control resides in the fact that these purine nucleotides combine with enzymes and alter the rate and direction of important control enzymes. For example, under stress the AMP/ADP ratio is increased and stimulates the activity of phosphofructokinase (PFK), thus accelerating glycolysis and stimulating ATP production. Isocitrate dehydrogenase is similarly stimulated by AMP, resulting in an accelerated Krebs cycle, with the ultimate regeneration of more ATP. These reactions are appropriate for survival since a significant fall in the concentration of ATP leads to increased AMP by the pathways ATP, ADP, AMP; the AMP mediates correction of the ATP level by stimulating glycolysis and phosphorylation. Similarly, when the demands on the cell are reduced, PFK is inhibited and reserves are conserved. Reactions of this type are normally taking place every instant with the result that ATP levels are maintained at optimal levels.

Another important aspect of this system is that evolutionary design has favored a high equilibrium constant over a large yield of ATP.

A high equilibrium constant allows the cell to advantageously use very small amounts of food while paying the price of a reduced energy yield. A higher yield would require a plentiful supply of fuel at all times, and this would reduce the potential for survival. However, since the efficiency of ATP production is relatively low, stressors and toxins have the effect of further reducing the ATP available for useful work and thus seriously reducing the cells' ability to carry out their functions. This is most readily seen in platelets which are unable to make ATP 'de novo' from its building blocks but must rely on scavenging and recycling the purine ring compounds such as adenine and hypoxanthine from its surroundings.

It is remarkable that the role of the adenine nucleotide pool is unique in being involved in virtually every metabolic sequence in the cell. The role of the adenylates is not specific to any single pathway but, more than any other compounds, they couple and correlate all the metabolic activities of the cell, giving rise to biological homeostasis and function. The effect of stress on this pathway is to reduce ATP and increase AMP and hypoxanthine (Figure 1). Therefore, we have developed methods to assay adenylate pools when platelets and white cells are brought into contact with a variety of chemicals.

METHODOLOGY

Because of the nature of the experimental conditions in which radioactive adenine was used as a precursor, the only radioactive compounds formed in significant quantities were ATP, ADP and AMP, with occasional formation of inosine monophosphate (IMP and HYPX). Dose response curves were used to express the results of known toxic compounds on adenylate metabolism as well as the effects of sewage, tap water and water effluents during treatment. In outline, the platelets were first obtained as a suspension by the standard method of differential centrifugation of anti-coagulated blood. Care was taken to prevent stressing the platelets during blood drawing or subsequently in the laboratory. Thus, only a 'clean' venipuncture from healthy blood donors was used and high speed centrifugation and washing avoided. Suspensions of white cells were prepared and treated similarly.

In a typical experiment, 5 μ l of U-14-C adenine (60,000 cpm) was added to a 0.5 ml suspension of platelets (5×10^5 /cu mm), monocytes or neutrophils (10,000/cu mm). Ten microliters of the test fractions to be evaluated were added. After incubation at 37°C for 30 minutes the cells were cooled in ice and centrifuged at 3000 rpm for 3 minutes. The supernatant was removed and 0.2 ml of cold 13% perchloric acid (PCA) was added to the pellet. After mixing and centrifuging, the PCA supernatant was analyzed by thin layer chromatography (TLC) and high performance liquid chromatographic systems (HPLC) described below. The TLC system allows one to measure the kinetic synthetic ability of the cells to convert U-14-C adenine to U-14-C purine nucleotides. The HPLC system measures the total pools of these substances.

THIN LAYER CHROMATOGRAPHY (TLC)

Ten microliters of the PCA supernatant were applied without heat to a 2-centimeter strip of an Eastman Kodak Cellulose Thin Layer Chromatogram #13255 with a mylar backing, and developed with a mixture of water: formic acid: tertamyl alcohol: 1:2:3 for 5 hours. The chromatogram was dried in air, cut into 15, 1-centimeter strips, and each placed in a 5-milliliter counting vial with 3 ml of scintillation fluid. The vials were counted in a scintillation spectrometer and each count calculated as a percentage of the total U-14-C nucleotide pool. A typical representation of the toxic effects of m-xylene is seen in Figure 2, with substantial decreases in ATP and ADP and an increase in AMP. Significantly greater amounts of unmetabolized adenine are seen in the cells exposed to m-xylene. Recovery is 95-102%. In a large series of normal platelets carried out in connection with other projects ATP was 65-70%, ADP was 10-14% and AMP was 1-5%. Changes of 15% or larger in ATP levels were regarded as being above the analytical and biological noise levels for this determination.

HIGH PERFORMANCE LIQUID CHROMATOGRAPHY (HPLC)

The major technical advance which makes it feasible to consider on-line capability is the use of high performance liquid chromatography (HPLC) for the rapid determination of the pool of adenine nucleotides at the rate of 15 minutes per sample. The peak heights are measured and used for calculating of pool size. Both TLC and HPLC are sensitive down to the 10^{-9} M range. With modifications the separation time can be reduced to 3 minutes.

A Waters & Associates HPLC apparatus with a U-18 micro-Bondapak reversed phase column was used. Fifteen microliters extract was injected and 0.1 M KH_2PO_4 was pumped through the column at 1.5 ml/minutes. The nucleotides and related compounds were adequately separated in 12-15 minutes and were detected by UV absorption at 254 nm (Figure 3). Identification was obtained using retention time and the addition of known compounds to the test PCA supernatant. In the future it is planned to pump the effluent from the HPLC system into a flow-through scintillation spectrometer so that specific activity of each compound can be computed, thus obviating the need for TLC. Recovery and reproducibility of the standards was 95-103%. The peak height was found to be linearly proportioned to concentration, regardless of peak width, in the range 10^{-6} to 10^{-12} moles or purine compound per sample injected. The perchloric acid was neutralized by the strong buffer used and did not interfere with the results or damage the columns.

RESULTS

The results are divided into three main sections:

- 1) Cellular effects of known toxic substances.
- 2) Effects of concentrated fractions of water.
- 3) Study of reverse osmosis permeate.

The chemical fractionation of various waters was carried out by Dr. Walton and is fully described elsewhere in this symposium.

1) Effects of Known Substances

Known compounds with the highest available grade of purity were added to the cell suspensions as described above. Initially, measurements were made of U-14-C adenine incorporation into ATP, ADP, and AMP, using TLC. Later on, when HPLC became available, the total adenylate pool was measured. A control with no additive was run with each batch of cells. Metabolic stress effects are indicated, either by a reduction in the 14-C or 12-C ATP pools, and similar results were obtained with each cell type (FIGS 4,5,6).

In general, the TLC and HPLC dose response curves were in good agreement (FIG. 5). The HPLC curves often showed a more sensitive response at the 0.1 and 1 ppm levels. The monocyte, however, was found to be most resistant to chloroform at concentrations of 0.1 and 1.0 ppm, responding at 10 ppm.³ Except for tetrachloroethylene the monocyte did not give a graded response for target cell release of chromium with increasing concentration. Thin layer chromatography and HPLC were more generally sensitive in detecting graded cell responses. These results were most satisfactory from a technical viewpoint and large numbers could be processed per day. In all the cells a concentration of added substance at 10 ppm was too high for the discrimination between individual compounds. The technique was more efficient at the 1 ppm level of exposure.

2) Effects of Concentrated Water Fractions

Concentrated fractions were produced and labeled from G1 to G9. The higher the G number the lower the polarity of the constituents of the fraction. Figure 7 shows the response of neutrophils to dilutions of each fraction. Due to the unavailability of reliable total organic values, an absolute concentration is not given for each fraction. Nevertheless, very significant responses were obtained with toxicity increasing with the G number of the fraction. Thus, G2 is seen to be relatively non-toxic to neutrophils, but G4, G5 and G9 produce significant reductions in the U-14-C ATP pool.

With regard to monocytes very similar results were obtained compared to those obtained on neutrophils. These results on extracts of Pomona, California wastewater note the changes in chromium release and U-14-C ATP pools. As before, toxicity generally increases with G number. Of special interest (FIG.8) is G4 and G4A; G4A is charcoal-treated in the plant and by comparison with the untreated G4 fraction it is seen that toxicity is reduced by charcoal treatment.

3) Reverse Osmosis

Concentrated samples of the permeate and reject supplied by Dr. H. Walton and toxicity determined as described above. FIG. 9. The G3 fraction of the reject was more toxic than its counterpart in the permeate.

DISCUSSION

The utilization of cell incubation followed by HPLC and TLC analysis offers considerable capability for the sensitive and rapid detection of cellular toxicity. The system is flexible in that it is not limited to any particular metabolic pathway or cell type and is relatively inexpensive. The system is vulnerable in that it needs a constant supply of normal cells of organisms. However, continuous 5-point dose response curves can be obtained using platelets from 10 ml of blood per day available from a nearby blood bank. Alternatively, one could use *in vitro* cell cultures of fibroblasts or bacteria which can be made up in large batches. If necessary, a portion of the cells can be diverted for functional studies or more detailed biochemical evaluation e.g. respiration, mitochondrial characteristics, enzymology etc.

Progress was also made with regard to problems of sensitivity of the assay, limited exposure time, and the selection of a model pathway. In general, sensitivity was adequate down to the 0.1 ppm range for known toxic compounds, and water concentrates of approximately 800 fold. It is estimated that by increasing the ratio of test substance to cells, an improvement of at least 10-fold could certainly be obtained and longer times of incubation could probably improve this by an addition factor of 2-5. With regard to the selection of a metabolic pathway, it is true that certain types of toxicity would be missed, for example, mutagenicity and very specific enzyme effects. Nevertheless, the adenylate pathway is of central and unique importance in the transduction of energy and the kinetic control of metabolism. Glucose, phosphate and DMSO had no significant toxic effects as measured by the present techniques. Thus, a major disturbance here would have serious consequences for any cell. More specific tests of toxicity could be done in response to a detailed knowledge of the chemical constitution of the components of the water concentrate.

Platelets have their limitations with response to the fact that they lack a nucleus and its DNA. For this reason, other cells need to be used when mutagenic characteristics are of interest. However, no serious problems are foreseen for the practical adaptation of these cells for continuous surveillance of metabolic toxicity of water effluents. The availability of platelets from small amounts of blood (1-5 ml/dose response curve), their stability and their quick response to perturbation makes them extremely suitable for our purpose. They are remarkably stable if kept suspended in their original plasma and not subjected to mechanical trauma. Neutrophils were more prone to deterioration due to technical reasons, and preparations had to be discarded on several occasions. Monocytes were very stable but their responsiveness to toxic materials present in concentrations below 10 ppm was not great.

It is difficult to decide which of these three cell models is the most useful for monitoring purposes. They all have their advantages and disadvantages. With on-line operation ultimately of prime importance, the platelet is recommended because of its stability and ease of analysis with only small amounts of blood (1-5 ml) needed per dose response curve. On the other hand, neutrophils and monocytes tell us more about the immune defense system, larger volumes of

blood on the order of 50 ml are required and the behavior of the neutrophils may be subject to seasonal variations.

Distinct differences were noted in the effects of wastewater before and after charcoal treatment. Both toxic and non-toxic fractions were found in the samples. Thus, the methodology had sufficient range to distinguish biologically between fractions of different chemical polarity. The humic acid fraction was non-toxic, but less polar compounds had adverse effects on the cells.

SUMMARY AND CONCLUSIONS

- 1) Several model systems for detecting cellular toxicity were developed using platelets, neutrophils, and monocytes.
- 2) Testing included known substances, water fractions, and concentrates from sewage and water treatment plants. Degrees of toxicity and the effects of purification were measured.
- 3) The metabolic pathway of adenylate metabolism was found to be suitable as a rapid method of analysis and allows for a unified, theoretical approach to toxicity of a wide variety of compounds. Toxic fractions detected in this manner can be reserved for more detailed biochemical investigation.
- 4) This technique is recommended for the detection of toxic compounds within hours of sampling and is thus applicable to modern water reuse technology.

Acknowledgements: Supported by EPA Grant #R803968-030. The monocyte experiments were performed by Drs. Weston and Norris. Technical and administrative help was provided by N. McDermott, R. Dustin, and K. Robbins.

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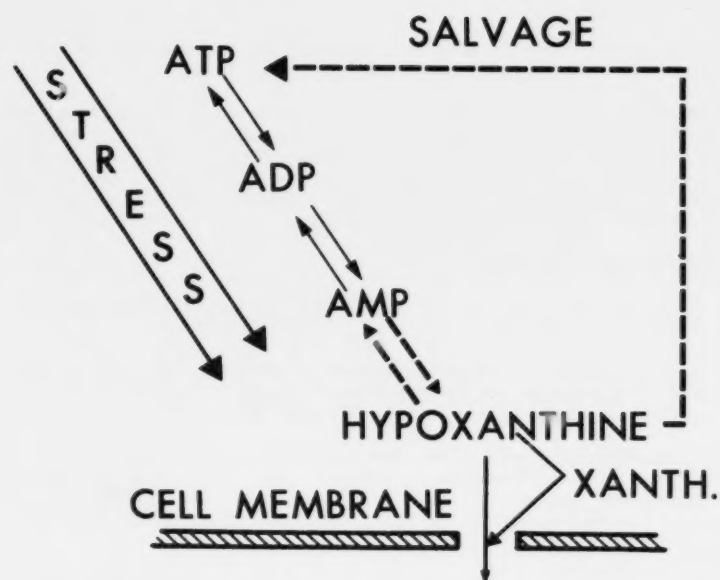


FIG 1 Partial nucleotide pathway indicating the effect of stress in changing the normal ratios of the intermediates.

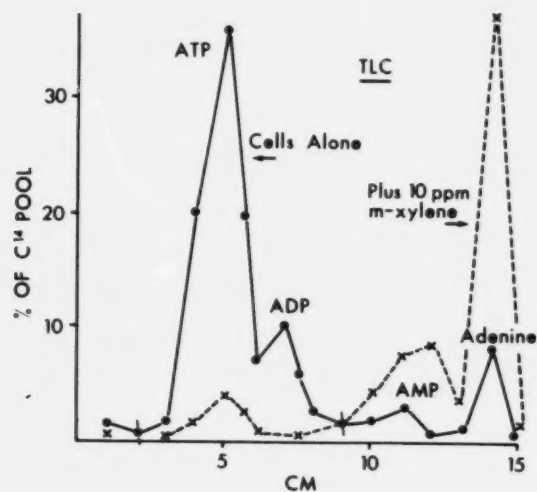


FIG 2 Synthetic rate of nucleotides from C^{14} Adenine affected by m-xylene. Synthesis is reduced and ratio of ATP+ADP/AMP is altered. This effect can result in inappropriate signals to cell enzymes.

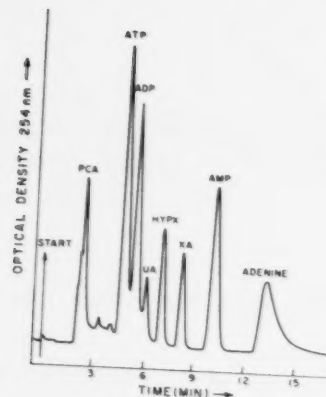
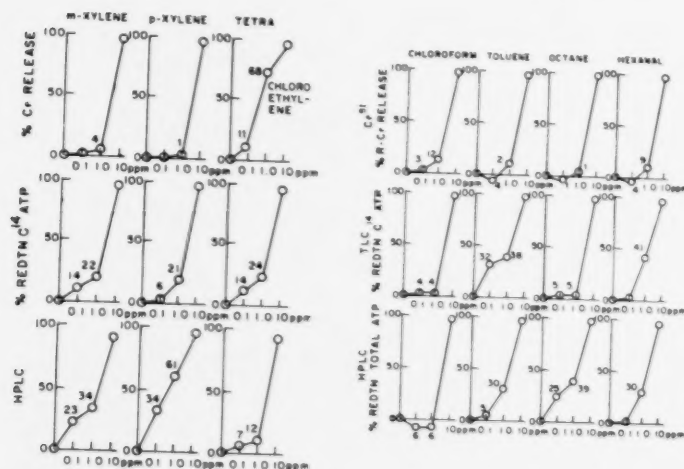


FIG 3 HPLC separation of purine-based compounds on C-18 microbondapak column.



FIGS 4 & 5 Comparison of percent reduction of neutrophil ATP pools, ATP synthesis and monocyte function due to known toxic compounds.

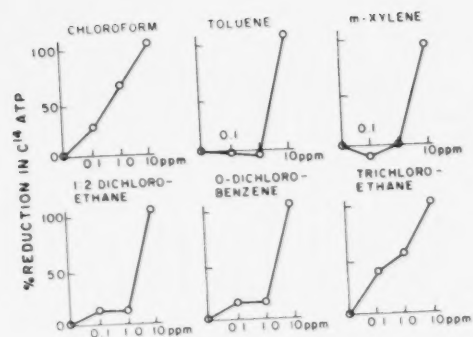


FIG 6 Reduction in platelet ATP synthesis due to known toxic compounds.

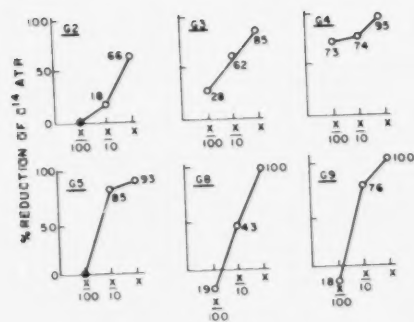


FIG 7 Reduction of cellular ATP by fractions isolated from water concentrates.

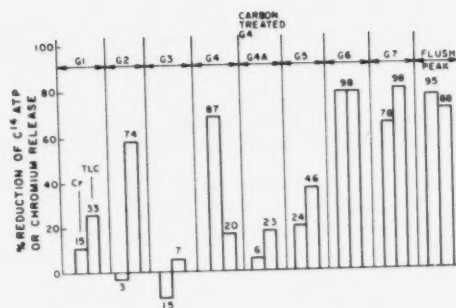


FIG 8 Reduction of cellular ATP and monocyte function in fractions obtained from water concentrates. Carbon treatment (G4A) significantly reduced the cellular toxicity seen in G4.

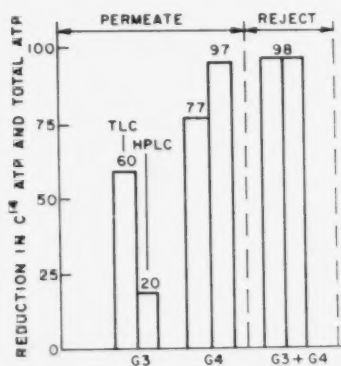


FIG 9 Reduction in cellular ATP pools (HPLC) and ATP synthesis (TLC) by fractions obtained from reverse osmosis permeate and reject.

HEALTH EFFECTS OF GROUNDWATER RECHARGE

Margaret H. Nellor
Project Engineer
Los Angeles County Sanitation Districts

Franklin D. Dryden
Head, Solid Waste Management Department
Los Angeles County Sanitation Districts

Ching-lin Chen
Supervising Civil Engineer
Los Angeles County Sanitation Districts

INTRODUCTION

In 1978 the Orange and Los Angeles Counties Water Reuse Study (OLAC) was formally established to develop a long range plan and implementation strategy for cost effective reuse of wastewater produced by the Orange/Los Angeles Counties region. Reconnaissance surveys conducted prior to initiation of OLAC identified some 400,000 acre-ft/yr (15.67 m³/s) of reuse potential within the study area. Of this total, approximately 60-70 percent was earmarked for groundwater recharge by means of surface spreading or deep well injection.

Groundwater recharge by surface spreading can generally be accomplished with a lower capital investment than other methods of water reuse because reclaimed water is commonly transported to spreading areas in existing storm water channels and less treatment is required. In Southern California, percolation sites are often adjacent to the major flood control channels used to convey storm runoff to the ocean. These spreading areas were originally designed to conserve storm flows, and are only used for that purpose for a few weeks or months each winter. The remainder of the year they are available for percolation of reclaimed water without additional capital expenditure.

California health officials have generally opposed expansion of on-going groundwater recharge projects and establishment of new projects based on concerns about unknown health effects. Although there is no documented evidence of any health problems associated with groundwater recharge programs, health authorities cite other factors as the basis for their concern. Authorities in the environmental health field have estimated that up to 90 percent of all cancer incidence results from human exposure to synthetic chemicals in the environment. There has been some concern that water supplies could serve as vectors for these cancer causing agents. Consequently, there is opposition to deliberately adding reclaimed water to potable water supplies and possibly affecting the health of consumers that utilize the supply. Although primary emphasis has been placed on the effects of trace organic constituents, additional health issues associated with water reuse by groundwater recharge include the potential effects of trace metals, minerals, and microorganisms that may be present in water supplies influenced by a groundwater recharge program.

In 1976, the State of California Water Resources Control Board, the California Department of Health Services, and the California Department of Water Resources convened a panel comprised of experts in the fields

of public health and environmental engineering and representatives of the public to discuss the health aspects of water reclamation by groundwater recharge. From discussions by the panel came health related research recommendations. These recommendations included: (1) that waters produced by an established, well engineered and operated groundwater recharge system be quantitatively analyzed to provide a data base for estimating health risks, and (2) that these quantitative analyses be coupled with epidemiologic studies of the populations consuming the water. The groundwater recharge programs in Los Angeles and Orange Counties provided an excellent opportunity to implement the recommendations of the panel, hence the Los Angeles County Sanitation Districts developed a proposal designed specifically to assess the health effects of water reuse by groundwater recharge. This proposal has since been incorporated into the overall OLAC work plan.

DESCRIPTION OF HEALTH EFFECTS STUDY SITES

The Orange/Los Angeles Counties Region encompasses three regional wastewater systems (the Los Angeles City System, the Los Angeles County Sanitation Districts' Joint Outfall System, and the Orange County Sanitation Districts' Regional Treatment System), each of which includes an extensive network of trunk sewers, treatment plants, and ocean outfalls. Each day, the 8.5 million people and industry located within the area generate a wastewater flow of 985 MGD ($43 \text{ m}^3/\text{s}$) which is processed by eleven existing wastewater treatment facilities. Within this overall study area, four sites were chosen for health effects research on the basis of historical or planned intentional water reuse by groundwater recharge. The four sites include the eastern section of the San Fernando Basin in the City of Los Angeles, the Anaheim Forebay and a coastal barrier injection site in Orange County, and the Montebello Forebay area in Los Angeles County.

San Fernando Basin

The San Fernando Valley lies in the western portion of Los Angeles County bounded on all sides by mountains and hills (see Figure 1). The Valley is drained by the Los Angeles River, which flows eastward along the Valley's southern boundary. Faulting and other geologic features have created impediments to groundwater movement which in effect divide the Valley into four separate groundwater basins. The San Fernando Basin is the largest of the four basins with a groundwater storage capacity of 3,000,000 acre-ft ($3.69 \times 10^{11} \text{ m}^3$) and a calculated safe yield of 90,680 acre-ft/yr ($3.55 \text{ m}^3/\text{s}$).

The eastern half of the San Fernando Basin is generally characterized as having materials of higher permeability and good water yields. These favorable geologic conditions have resulted in significant development of the groundwater resources in that area. The eastern portion of the basin contains a majority of the producing groundwater recharge facilities used to spread native runoff and water imported from the Owens Valley. The cities of Los Angeles, Glendale and Burbank operate well fields in the San Fernando Basin which are used for domestic water supply. Groundwater extraction by the City of Los Angeles is approximately 86,300 acre-ft/yr ($3.38 \text{ m}^3/\text{s}$) while those of Glendale and Burbank are approximately 4,700 acre-ft/yr ($0.18 \text{ m}^3/\text{s}$) each.

As indicated in Figure 2 there are six spreading ground areas in the San Fernando Basin operated by the Los Angeles County Flood Control District

and by the City of Los Angeles. These spreading facilities have an estimated infiltration capacity of 396 acre-ft/d ($5.66 \text{ m}^3/\text{s}$) on a short term basis. Some incidental groundwater recharge occurs by discharging unused effluent from two operating wastewater treatment facilities into the unlined section of the Los Angeles River. Future plans for groundwater recharge in this area include the use of effluent from these facilities as well as the proposed Sepulveda Water Reclamation Plant.

Anaheim Forebay

The Orange County Groundwater Basin includes over 200,000 acres (80,939 ha) drained by the Santa Ana River, which originates northeast of Orange County in the San Bernardino Mountains and flows southwesterly through Orange County to the ocean. (See Figure 3). The basin has approximately 40,000,000 acre-ft ($4.9 \times 10^{10} \text{ m}^3$) in storage and an annual average overdraft (difference between extraction and natural replenishment) of 95,000 acre-ft ($1.17 \times 10^9 \text{ m}^3$). Currently, about 60 percent of all water used in Orange County or some 243,500 acre-ft/yr ($9.54 \text{ m}^3/\text{s}$) is supplied from groundwater.

Within the basin, the Anaheim Forebay area is characterized as having the highest rate of permeability and contains the major groundwater replenishment facilities. The Orange County Water District, the agency responsible for management of the groundwater basin, owns and utilizes approximately 730 acres (295 ha) of the Santa Ana riverbed and 300 acres (121 ha) adjacent to the river channel for ponding imported water (Colorado River water and California State Water Project water), storm flows, and base flows of the Santa Ana River. Facilities are designed to percolate up to 300,000 acre-ft/yr ($11.75 \text{ m}^3/\text{s}$). Some incidental reuse has occurred via discharge of approximately 750,000 acre-ft ($9.23 \times 10^9 \text{ m}^3$) of reclaimed water from San Bernardino and Riverside Counties into the Santa Ana River. Future plans for groundwater recharge include construction of an advanced wastewater reclamation plant in the Anaheim Forebay which would supply up to 44,820 acre-ft/yr ($1.76 \text{ m}^3/\text{s}$) for replenishment by surface spreading.

Orange County Coastal Barrier

In addition to groundwater replenishment via surface spreading, the Orange County Water District is responsible for a coastal barrier injection program designed to prevent seawater intrusion and augment potable groundwater supplies (See Figure 3). The barrier system consists of a series of injection wells which deliver fresh water to underground aquifers to block further passage of seawater, and a series of extraction wells which intercept brackish water and return it to the ocean. The injected water is a blend of deep well water, reclaimed water and desalted wastewater supplied by Water Factory 21. Major facilities of Water Factory 21 include a 16,800 acre-ft/yr ($0.66 \text{ m}^3/\text{s}$) advanced wastewater reclamation plant (chemical clarification, ammonia stripping, recarbonation, multi-media filtration, carbon adsorption, chlorination) and a 5,600 acre-ft/yr ($0.22 \text{ m}^3/\text{s}$) reverse osmosis demineralization plant. Pilot studies have been underway since the inception of Water Factory 21 to determine the fate of the reclaimed water injected into groundwater supplies.

Montebello Forebay

The Coastal Plain of Los Angeles County is subdivided into two major groundwater basins (the Central Basin and the West Coast Basin) by a

fault zone which acts as an impediment to groundwater movement. The Central Basin is commonly divided into four subdivisions: the Los Angeles Forebay, the Montebello Forebay, the Whittier area and the Central Basin Pressure area. The two forebay areas and the Whittier area occupy the northerly portion of the Central Basin adjacent to the Santa Monica Mountains, Repetto Hills, and Puente Hills, and the Pressure area occupies the southerly portion of the basin (see Figure 4).

Groundwater enters the Central Basin through surface and subsurface flow and by direct percolation of precipitation, stream flow, and applied water. The main surface and subsurface flow into the basin is through the Los Angeles Forebay and Whittier Narrows from groundwater basins in the interior valleys. Replenishment of the aquifers occurs in the forebay areas where permeable sediments are exposed at the ground surface.

In the Los Angeles and Montebello Forebays, aquifers are, in general, in hydraulic continuity with each other and with the ground surface. The areas of contact between aquifers and the areas of contact with the ground surface are important because it is only through these areas that supplemental water can be introduced by spreading into the aquifers in large quantities. In this regard, the most important area in the Central Basin is in the vicinity of the Whittier Narrows in the Montebello Forebay area because of the interconnection of the deeper aquifers through the shallower aquifers to the ground surface.

Total capacity to store groundwater in the Central Basin above the base of the deepest aquifer is approximately 13,800,000 acre-ft ($1.70 \times 10^{10} \text{ m}^3$). Since 1904 utilized storage in the Central Basin has amounted to approximately 780,000 acre-ft ($9.59 \times 10^8 \text{ m}^3$). Corresponding figures for the Montebello Forebay are 1,800,000 acre-ft ($2.21 \times 10^9 \text{ m}^3$) and 190,000 acre-ft ($2.34 \times 10^8 \text{ m}^3$) respectively.

Replenishment of Groundwater Supplies in the Montebello Forebay

Under natural conditions, the groundwater in the Montebello Forebay was replenished by: 1) Percolation from the channels of the San Gabriel River; Rio Hondo and other streams crossing the area. The flow in these streams included storm runoff and rising water from groundwater in the San Gabriel Valley; 2) Subsurface flow from the San Gabriel Valley into the Montebello Forebay through the Whittier Narrows; and 3) Percolation of direct precipitation on the Montebello Forebay area.

Since the 1920's, the Los Angeles County Flood Control District has artificially replenished Montebello Forebay groundwater supplies by constructing dams and reservoirs in the mountainous area tributary to the San Gabriel and Rio Hondo Rivers, releasing the captured storm flows at rates within the percolation capacity of the river channels, and constructing spreading areas which allow for more rapid percolation of storm flows. During this period, the San Gabriel River Channel has been modified to increase percolation rates and spreading basins have been constructed adjacent to the Rio Hondo and San Gabriel Rivers.

The stream channels and offstream spreading basins are only utilized a small portion of the year for percolation of storm waters. When not used for storm waters, the basins have been utilized for spreading imported waters since the early 1950's, when Colorado River supplies were first purchased from the Metropolitan Water District of Southern California.

Beginning in 1962, reclaimed water from the Whittier Narrows Water Reclamation Plant (WRP) of the Los Angeles County Sanitation Districts became available for spreading. Chlorinated secondary effluent from Whittier Narrows WRP was purchased by the Central and West Basin Water Replenishment District and spread in Flood Control District groundwater recharge basins. At present, three County Sanitation Districts' WRP's produce reclaimed water used for groundwater recharge in the Montebello Forebay: Whittier Narrows WRP, San Jose Creek WRP, and Pomona WRP. Since 1962 the Whittier Narrows facility has produced approximately 235,000 acre-ft ($2.89 \times 10^8 \text{ m}^3$) for groundwater recharge while Pomona WRP has contributed approximately 68,000 acre-ft ($8.36 \times 10^7 \text{ m}^3$) since 1968, and San Jose Creek WRP has contributed approximately 51,000 acre-ft ($6.27 \times 10^7 \text{ m}^3$) since 1972. The Whittier Narrows and San Jose Creek WRP's now include dual-media filtration systems designed to facilitate disinfection. The Pomona Plant has employed activated carbon filtration since 1977.

The present recharge practice includes dilution of reclaimed water with imported and storm waters in a two to one ratio (i.e., two parts imported and storm water to one part reclaimed water) to provide for a groundwater with less than 10 mg/l NO_3 . The effluent from the Whittier Narrows WRP is spread chiefly in the Rio Hondo spreading grounds (See Figure 4). Effluent from San Jose Creek WRP is conveyed through a closed pipeline to a diversion point close to the San Gabriel River Spreading Grounds where historically it has been spread both in the spreading grounds and the unlined reaches of the San Gabriel River. Not all of the effluent produced by San Jose Creek WRP has been used in the past for recharge because of opposition to expansion by the State Health Department and because, at times, the total dissolved solids concentration exceeded the Replenishment District's specified limit of 700 mg/l. Approximately 15 percent of the reclaimed water produced by the Pomona WRP is reused for irrigation purposes in the Pomona area. The remaining flow is discharged into the concrete lined section of San Jose Creek in Pomona and conveyed to unlined portions of San Jose Creek and the San Gabriel River where it infiltrates near the Whittier Narrows Dam.

Between July 1962 and September 1978, the quantity of reclaimed water percolated into potable groundwater aquifers totaled 354,000 acre-ft ($4.35 \times 10^8 \text{ m}^3$). By comparison, the total recharge to the Montebello Forebay (including stormwater, imported water, reclaimed water, and subsurface flow) has been 3,100,000 acre-ft ($3.81 \times 10^9 \text{ m}^3$). Therefore, over the last 16 years reclaimed water has comprised 11 percent of the total water input. On an annual basis the highest reclaimed water input percentage has been 23 percent during the drought years of 1975-76.

The Rio Hondo and San Gabriel River spreading grounds have total areas available for percolation of 455 and 101 acres (184 and 41 ha), respectively. Approximately 133 additional acres (54 ha) are available in the unlined portions of the San Gabriel River. The spreading grounds are subdivided into numerous individual basins ranging in size from 4 to 20 acres (1.6 to 8.1 ha). A basin filling cycle has been established that serves several purposes, including maintenance of aerobic conditions in the upper soil strata to improve treatment and prevent sealing of the basins and to control vector insects. Batteries of basins are rotated through a 21 day cycle consisting of filling to a depth of 4 ft (1.22 m) for seven days, draining for seven days, and drying for seven days. On this rotating basis, the capacity of the basins is approximately 224,100 acre-ft/yr ($8.78 \text{ m}^3/\text{s}$). On a short term basis with all

the basins in operation simultaneously, the capacity for groundwater recharge is approximately 672,000 acre-ft/yr (26.33 m³/s). This latter mode of operation is used during storm periods to maximize conservation of storm water.

Montebello Forebay Monitoring Program

Wells in and adjacent to the Montebello Forebay produce a substantial quantity of groundwater for distribution by domestic water companies along with surface deliveries of imported water. The State Department of Water Resources has a significant amount of historical water quality data on most of these wells consisting mainly of standard mineral and bacterial analyses. In 1972, the County Sanitation Districts, Flood Control District and Replenishment District embarked on a joint program of increased analyses of the different sources of water used to replenish the Montebello Forebay, as well as an intensive program of monitoring a network of 16 wells within the forebay. In general this program consists of quarterly analyses for major mineral constituents and physical properties, and annual analyses for trace constituents such as heavy metals and pesticides. To date, the monitoring program has demonstrated that the well water consistently meets drinking water standards.

HEALTH EFFECTS STUDY OBJECTIVES

The County Sanitation Districts have developed a work plan for evaluating the health aspects of water reuse via groundwater recharge based on research recommendations made by the State Water Resources Control Board's Consulting panel. The main objective of this health effects study will be to determine whether the historical level of reuse in the Orange/Los Angeles Counties area has caused any discernible health effects. The data generated by this assessment can be used by California regulatory authorities as a basis for making decisions regarding continuation, modification, or expansion of existing groundwater recharge projects and initiation of new projects. These data will also be used for developing groundwater recharge criteria contingent upon such factors as the type of wastewater treatment provided, effluent quality and quantity, spreading or injection operations, soil characteristics, hydrogeology, residence time, and distance to withdrawal.

The overall study plan includes four subtasks: 1) an assessment of the influence of reclaimed water on potable groundwater supplies, 2) water quality characterizations, 3) toxicologic characterizations, and 4) epidemiologic studies. The first three subtasks comprise Task 1 of the Orange and Los Angeles Counties Water Reuse Study with full funding being supplied by the Reuse Study. Epidemiologic studies are being funded separately from the Reuse Study by research grants from the Environmental Protection Agency and the California Department of Water Resources, but will be coordinated with Task 1 work and results. The first and fourth subtasks are being conducted exclusively in the Montebello Forebay because this area historically has the most significant level of reuse. Water quality and toxicologic characterizations are being conducted for all four sites.

THE INFLUENCE OF RECLAIMED WATER ON POTABLE SUPPLIES

One factor required in assessing the health effects of groundwater recharge is an estimate of the percentage of reclaimed water in the water supplies of an exposed population. In the case of the Montebello Forebay, reclaimed water becomes a part of the drinking water supply by

inclusion with imported water and storm waters used for groundwater replenishment. As this mixture percolates through the soil, many of the reclaimed water constituents suspected to be of health significance may be mitigated as a result of physical, chemical, or biological reactions that occur during percolation. With or without modification, this mixture eventually becomes a part of the groundwater, where further dilution may be achieved as it moves through a series of aquifers and is subsequently pumped from wells located throughout the forebay.

The two principal sources of water supply for the water systems serving the Montebello Forebay and adjoining areas are groundwater and imported water purchased from the Metropolitan Water District of Southern California. Within this total service area, the relative use of well water compared to imported supplies and/or the relative use of water pumped from different wells varies from location to location, and may even vary within individual water districts.

In order to quantify the effects of reclaimed water on potable groundwater supplies, it is first necessary to determine the quantities of reclaimed water pumped from Montebello Forebay wells at any point in time. The most promising approach to this problem involves the use of a previously developed water movement model for the Montebello Forebay based on sulfate as a tracer. This procedure is founded on the fact that sulfate concentrations in Colorado River water are substantially greater than sulfate concentrations in native groundwater or in any of the other sources used for replenishment. Based on knowledge of the historical proportions of Colorado River water making up the total amount of water spread and knowledge of the change in sulfate ion concentration in well water over time, it is possible to mathematically define the percentage of Colorado River water in the water pumped from any particular well. It has been assumed that the percentage of Colorado River water in each well related to the total amount spread will be in the same proportions for reclaimed water.

The sulfate model does have some limitations. The model is unable to account for such factors as the variability in the proportions of Colorado River water spread from year to year and among the different sections of the spreading grounds, variability in the concentration of sulfate ion in the native groundwater, and variability in pumping rates over time. It is also no more reliable than the historical sulfate data available. In view of these limitations, it is possible that estimates of reclaimed water reaching wells derived from the sulfate model could differ from the true amounts by 15 to 20 percent.

The reliability of these estimates would be greatly enhanced if there were a tracer in reclaimed water which, when found in well water, would in itself give a direct indication of the proportion of reclaimed water reaching individual wells. Two potential tracers, bromide and coprostanone, have been designated for further investigation during the early stages of the study. This assessment will determine whether these compounds can survive biological treatment, chlorination, and percolation and if their concentrations in reclaimed water are significantly higher than concentrations in imported and storm water, and native groundwater. If either tracer meets these criteria, well water data will be collected and used to verify sulfate ion model results and possibly to adjust calculations derived from the model.

Information on estimates of reclaimed water reaching Montebello Forebay wells will be combined with results from analyses of water supply and

water delivery practices among individual water purveyors within the forebay to derive realistic estimates of percentages of reclaimed water contained in water delivered to domestic consumers. The end product will result in the compilation of maps for years 1962 through 1976 depicting percentages of reclaimed water delivered by individual water district. These maps will be used by the epidemiologic researchers to better define their study area boundaries and segregate certain population groups by exposure rate for more intensive investigation.

One additional objective of this subtask will be to investigate water quality changes that occur in reclaimed water as it percolates through soil. Previous studies have demonstrated that most of the contaminants present in reclaimed water are substantially reduced during vertical percolation through the unsaturated zone, as well as during horizontal movement in the aquifer. ¹⁻⁸ Chemical oxygen demand (COD) measurements indicate substantial removal of COD during percolation. Notable exceptions are some heavy metals, total dissolved solids, hardness and some trace organics. Although these studies have contributed to the understanding of changes in reclaimed water during percolation, the fate of many constituents, in particular heavy metals and trace organics, is still unknown. To aid in the understanding of the effects of percolation, pilot scale percolation studies are planned during the Health Effects Study. This plan includes construction of a test basin in an area of similar soil composition to that of the Montebello Forebay spreading ground area and adjacent to an existing test basin site used for earlier percolation studies. To insure minimum soil disturbance at the site, porous cup soil moisture samplers will be installed at depths of 2, 4, 6, 8, and 10 ft (0.6, 1.2, 1.8, 2.4, 3.0 m) to collect soil water samples. Test basin operation will simulate the existing spreading operation's filling, draining and drying regimes. Undiluted tertiary effluent samples from the Whittier Narrows WRP will be the source of reclaimed water spread. Percolates will be collected and analyzed for metals, minerals, nitrogen compounds, gross organics, and possibly specific trace organics depending upon the volume of sample extracted.

Although only a simulation, this study should provide an overview of the fate of many health significant constituents during percolation and the capacity or capability of a certain soil type(s) to remove contaminants. Additional plans call for performing soil column studies using soil cores from the San Fernando Basin and the Anaheim Forebay to estimate the removal capacity of the other recharge sites within the Orange/Los Angeles Counties area.

WATER QUALITY CHARACTERIZATION

The objective of this subtask is to produce detailed water quality characterizations of well water, reclaimed water, and other sources of water used for replenishment and thereby develop a water quality base for assessing health risk. These data will also relay significant information to the epidemiology subtask in terms of population exposure to health significant waterborne constituents. Initial groundwater sampling is being conducted in the Montebello Forebay, the San Fernando Basin, the Anaheim Forebay, and the Orange County coastal barrier site. After control (unexposed) populations are selected for the epidemiology study the water quality characterization program will be expanded to include these areas. Reclaimed water sampling includes Whittier Narrows, San Jose Creek, Pomona, Los Angeles-Glendale, and Water Factory 21 WRP's. Sampling of additional recharge sources includes imported

Metropolitan Water District water, natural flows of the Los Angeles and Santa Ana Rivers, and storm flow when available.

The water quality characterization program involves analyses for inorganic and organic constituents as well as microorganisms. Inorganic characterizations include traditionally monitored minerals, nutrients, and metals while organic sampling includes both gross organic and specific trace organic characterizations (See Tables 1 and 2). A list of health significant trace organic compounds has been compiled which includes several classes of organics that may act as indicators of the impact of reclaimed water on groundwater supplies. This list was developed from lists compiled by the Environmental Protection Agency, California Department of Health Services, National Academy of Sciences, and the Occupational Safety and Health Administration which enumerate individual organics of known or suspected health significance. Many of these same compounds have been found in the effluents of wastewater treatment plants operated by the Sanitation Districts of Los Angeles County and the Orange County Water District; hence, there is some concern that compounds have entered the groundwater system as a result of percolation and/or injection activities. Trace organic samples will be collected and processed using a high pressure-high volume trace organics concentrator. This system consists of four macroporous resin columns in series, a high pressure pump, and a UV detector. The system is capable of processing a 26.5 gal (100L) sample at a flow rate of 0.13 gpm (0.5L/min) at a pressure of 1500 psi (105.5 kgf/cm²) and achieving 90 percent recovery of dissolved organics (See Figure 5). Chemical analyses will be accomplished by combined gas chromatography/mass spectrometer/computer methods (GC/MS/COMP).

During the course of the study, decisions will be made to add or remove compounds from either the inorganics or trace organics list which are (a) not detected or detected too infrequently to justify further testing, (b) frequently isolated but not on the list, or (c) found to be of interest as a result of studies from the other Health Effects Study subtasks. Adjustments in sampling frequency will also be made as data are collected and reviewed.

Microorganism characterizations will include both viruses and bacteria. The County Sanitation Districts have developed a truck-mounted virus concentrator capable of processing up to 1000 gal (3789 L) of tap water and 500 gal (1894 L) of wastewater in a reasonable time frame. As indicated in Figure 6, the unit is a flow-through membrane adsorption system which incorporates automatic pH adjustment and aluminum addition during sample collection. Developmental experiments have included sampling runs using seeded polio, coxsackie, ECHO, and adeno viruses, evaluations of disinfection procedures for the sampling unit, and sampling final effluents from some of the County Sanitation Districts' treatment plants as a trial run to work out field problems prior to initiation of the study. Initial virus sampling will include treatment plants only. Potable samples will be processed after completion of a clean room facility to be used only for assaying clean water samples and thereby eliminating the chance of contamination by wastewater or seeded samples. In addition to virus sampling, standard bacteriological water quality tests (total and fecal coliform, fecal streptococci, and standard plate count) will be performed.

TOXICOLOGY

The goal of the toxicology subtask is to assess the impact of the trace organic content of reclaimed and surface waters on groundwater quality.

This shall be accomplished in three ways. First, well water, reclaimed water, and surface water are being surveyed for the presence of trace organic compounds and classes of compounds of known or suspected health significance. This work is categorically part of the water quality characterization subtask but results will be coordinated with the toxicology effort. Second, cellular bacterial and mammalian systems which are highly sensitive to mutation and other morphological changes are being used as screening tests to identify those samples and fractions of samples containing health significant organics. Fractions will be analyzed in detail in order to identify those components eliciting toxic responses. Once identified, these materials may be traced to replenishment sources based on water quality characterization information and impacts on water quality evaluated. Third, the dose response behavior of the cellular testing systems may be used to evaluate the relative toxicity of the types of water sampled.

Because of the difficulty involved in attempting to characterize complex mixtures of organic constituents in water and wastewater coupled with the expense and effort involved in traditional animal toxicity testing, a simple strategy for investigating the carcinogenic activity of discrete fractions of water concentrates was devised. The Ames Salmonella test and the mammalian cell transformation test are the key mechanisms in the toxicology experimental design. Both test systems have been used successfully in identifying and detecting known and suspected carcinogens, and have been demonstrated to be rapid, sensitive, and relatively inexpensive tools for screening environmental samples as compared to traditional toxicity testing. The Ames Salmonella/Mammalian-Microsome Mutagenicity test uses several strains of specially developed mutants of Salmonella typhimurium which can be reverted from a form which requires the amino acid histidine for growth back to histidine independence by a wide variety of mutagens. The Ames test has also been shown to be 90 percent accurate for detecting carcinogens as mutagens. The mammalian cell transformation test is a procedure in which normal cultures of cells or cell lines (such as mouse cells) are dosed with carcinogens, resulting in morphological transformation or malignancy in genetic proliferata. Additional toxicologic information can be supplied by the mammalian cell transformation including the presence of cancer promoting agents and in vivo (animal testing) of cells transformed in vitro.

In general, the toxicologic screening procedure involves the following steps as indicated in Figure 7: (a) Total organic residues from samples collected and processed for trace organic water quality characterizations will be subjected to Ames testing and mammalian cell transformation testing; (b) If a positive mutagenic response is elicited, the organic concentrate will be fractionated into simpler mixtures using high pressure liquid chromatography and the fractions then separately applied to the cell cultures; (c) Any fraction yielding a positive mutagenic response will be further fractionated into even simpler mixtures for eventual GC/MS/COMP identification. This iterative fractionation/ toxicity testing is designed to remove as many insignificant components from the complex organic concentrate as possible so that the task of identifying health significant components will be more manageable.

There are, however, some limitations in this experimental design. For example, the trace organics concentrator will not be capable of capturing 100 percent of the organic material present in a sample, despite the fact that the technology used in developing the concentrator is state-of-the-art. This problem is being circumvented by installation of rigid

quality control checks such as continuous monitoring of the concentration breakthrough, total organic carbon analyses of processed samples and sample effluents, and further concentration and Ames testing of sample effluents that demonstrate component breakthrough. Additional limitations are created by the fact that many health significant compounds, once concentrated, may not be identified and/or quantified with present analytical methodologies.

Although there remains considerable doubt as to whether tests with concentrated samples are relevant to human exposure at low dosages, the data from this study are not intended for use in this way. These results cannot be, nor are they intended to be, correlated or extrapolated to human health statistics. Despite these limitations, the Health Effects Study toxicology effort will provide a body of knowledge derived from observations of biological effects and chemical data obtained from actual samples, which can, when coupled with results from other elements of the Study, be used as a basis for evaluating the impacts of groundwater recharge.

EPIDEMIOLOGY

The need for population-based data for assessing the health effects of groundwater recharge mandates an epidemiologic component in the Health Effects Study. The Montebello Forebay has been selected as the general study area based on length of exposure to reclaimed water via an ongoing groundwater recharge program. The epidemiology study will evaluate a variety of health data sources to determine if the population which has been consuming groundwater replenished in part by reclaimed water exhibits unusual patterns of disease occurrence relative to concurrent unexposed populations. The Study will be performed by researchers at the School of Public Health, University of California at Los Angeles in conjunction with the California Department of Health Services and the Los Angeles County Sanitation Districts.

The epidemiology study is divided into two phases: a six-month planning period and a twenty-four month field work, data gathering, and analysis phase. The specific purpose of the planning period is to design a study protocol for the second phase of work. The emphasis will be upon retrospective comparisons of a wide variety of available health and illness indices among exposed and unexposed communities. The initial investigation will evaluate a wide spectrum of diseases rather than giving special priority to one type of illness. This will enable investigators to detect unanticipated health effects due to unknown causes as well as illnesses due to exposure to known agents. Data are to be examined during the planning period with respect to their usefulness, validity, and availability. Types of data to be assessed include mortality data by census tract, cancer registry data, fetal and infant deaths, congenital malformations, hospital discharge records, school absenteeism records, and data from infectious disease reporting systems. The demographic and sociocultural aspects (age, sex, ethnic distribution, occupation, migration pattern, income, education, population density, housing quality) of the exposed study population will also be examined to aid in selecting comparison or control populations. The second phase of the study will be devoted to acquiring the necessary data sets to carry out the investigation and to develop the epidemiological and statistical approaches necessary to conduct the analyses.

Disease in communities is a reflection of a vast and complex interaction of social and biological factors and the available data about both

the disease and environmental factors are often imprecise. In this case, a major environmental influence, the water source, is being impacted in such a way that it could become a vector of both known and unknown disease producing agents. If this change were associated with acute health problems of great consequence, then by now these health problems would be obvious. The real problem is identifying those diseases which have a long lag period between exposure and clinical illness, and those diseases which are not serious enough to make people seek medical attention. The usefulness of this study can be limited by these and other problems including: (1) The problem of confounding variables--individual characteristics such as smoking, diet, and occupation modify the risk of disease. These characteristics, often related to social class and ethnic group, can bias investigations which involve examinations of existing data; (2). The problem of accurate information about the population at risk--basic demographic data which will be used to choose study and control populations will come from the 1970 census. The high rate of residential mobility in the Los Angeles area is well documented and it is quite possible that the census is, by now, inaccurate. These same census data will be the "denominators" for calculating rates of disease for comparisons and, if in error, could greatly effect the analyses; (3) The problem of a long incubation period between exposure and clinical disease--there is evidence that as much as 15 to 20 years may pass between the exposure and manifested illness for some types of cancer and other chronic diseases. With the high rate of migration, people could be exposed in one area and move to another before becoming ill, and thus the cause and effect association would be missed; (4) The problem of small numbers of disease events--the exposed population may be less than 100,000. When rare disease events are examined separately by sex and ethnic groups, it may be impossible to detect meaningful differences; (5) The lack of specific hazard--the lack of any identified hazard or specific effect makes the identification of risk or the substantiation of no risk much more difficult; (6) The problem of accurate information on population exposure--limitations in computing exposure data have been previously described under subtask 1. This problem is complicated by the fact that significant utilization of bottled water is known to occur in the Los Angeles area which will make detection of true health effects more difficult.

While these problems are real, they do not outweigh the need for an epidemiologic study. They do, however, indicate the need to concentrate on health indices which would be sensitive to the situation and to consider that the final result of this effort may be inconclusive. It is also important to reiterate that results from an individual subtask such as the epidemiology study cannot stand alone, but must be combined with the data generated by the other elements of the study to supply an overview of all health aspects of groundwater recharge.

TIME TABLE AND COSTS

The starting date of the three-year Health Effects Study was November 1978. A timetable for the study is presented in Figure 8. Check points have been built into the study to allow funding authorities to evaluate progress and results prior to commitment of additional funds and to allow for mid-course corrections in the study design. Check point intervals have been set at six months for Reuse Study Task 1 work and at one year for the Epidemiology Study. The budget for the entire Health Effects Study is presented below:

<u>SUBTASK</u>	<u>COST</u>
Influence of Reclaimed Water on Potable Supplies	\$ 132,000
Water Quality Characterization	306,000
Toxicology	592,000
Epidemiology	<u>400,000</u>
TOTAL COST	\$1,430,000

DISCUSSION

In light of the formidable technical problems facing the health effects study, one could ask: Why attempt such a study when the odds on detecting any health effects are so low--much less relating the effect to a specific causal agent? The answer is in part philosophical and pragmatic. Agencies which process wastewater and regulate or determine its use as a supplemental water supply, owe the public that utilizes that supply the best practical scientific evaluation of the health risk that might be associated with it. As one looks to the future, expanded potable water reuse may be a proper way of meeting man's need for more and more water. Others fear that renovated wastewaters contain concentrations of chemical and biological components that will eventually cause illness to those exposed even after extensive surface treatment and percolation through fine-grained aerobic soils.

Regulatory agencies recognize that neither the optimists nor the pessimists have any significant data upon which to base their case. Although it is impossible to prove a negative, a careful scientific look at all health aspects of a planned groundwater recharge project such as that of the Montebello Forebay will be useful even if the results prove to be inconclusive. The absence of any positive correlation between reuse and health effects should provide some relief for those concerned that there may be serious undetected health effects occurring now. Furthermore, the absence of such positive findings should provide the rationale for public health agency approval of some modest expansion of recharge programs in existing sites or in new sites with similar characteristics, in which case the value of the study may be more obvious. Until such findings are developed, it is useless to speculate on how the data would be used to modify treatment or recharge programs to again reduce the health risk. In either event, perhaps water reuse projects will at least begin to get over the hurdle that is always placed in front of new proposals: "How do you know there isn't a health problem when no one has ever taken a good look?"

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TABLE 1. LIST OF CONSTITUENTS FOR WATER QUALITY
CHARACTERIZATIONS

<u>PARAMETER</u>	<u>METHOD OF ANALYSIS</u>
Alkalinity	Titration
Arsenic	Atomic Absorption
Barium	Atomic Emission
Cadmium	Atomic Absorption
Chromium	Atomic Absorption
Chemical Oxygen Demand	Dichromate Reflux
Color	Nessler
Copper	Atomic Absorption
Cyanide	Distillation
Hardness	Titration
Iron	Atomic Absorption
Lead	Atomic Absorption
Manganese	Atomic Absorption
Mercury	Atomic Absorption
Nickel	Atomic Absorption
Ammonia-Nitrogen	Distillation-Titration
Nitrate-Nitrogen	Cadmium Reduction
Nitrite-Nitrogen	Colorimetric
Phosphate	Ascorbic Acid
pH	ph Meter
Selenium	Atomic Absorption
Silver	Atomic Absorption
Sodium	Atomic Emission
Sulfate	Turbidimeter
Total Dissolved Solids	Filtration-Evaporation
Total Organic Carbon	TOC Analyzer
Trace Organics (See Table 2)	GC/MS/COMP
Turbidity	Hach Turbidimeter
Zinc	Atomic Absorption

TABLE 2. LIST OF SPECIFIC TRACE ORGANIC COMPOUNDS
GC/MS/COMP ANALYSIS USING PURGEABLE AND
NON-PURGEABLE TECHNIQUES

Purgeable Organics

Vinyl chloride	Trichloroethylene
Methylene chloride	Dibromochloromethane
1,1-dichloroethane	Benzene
Chloroform	Bromoform
1,2-dichloroethane	Toluene
Carbon Tetrachloride	Chlorobenzene
Bromodichloromethane	Dimethylnitrosamine
o-dichlorobenzene.	Di-(2-ethylhexyl)phthalate
p-dichlorobenzene	Lindane

Non-Purgeable Organics

1,2,4-trichlorobenzene	BHC
Trichlorophenols (isomers)	Endrin
Pentachlorophenol	Aldrin
Bis(2-chloroethyl)ether	Dieldrin
Fluoranthene	DDT
3,4-benzpyrene	DDE
Benzo(g,h,i.) perylene	DDE
Indeno(1,2,3-c.d.)pyrene	Tetrachlorobiphenyls
Bis-chloromethyl ether	Pentachlorobiphenyls
3,4-benzfluoranthene	Hexachlorobiphenyls
Hexachlorobenzene	
Bis-chloromethylether	

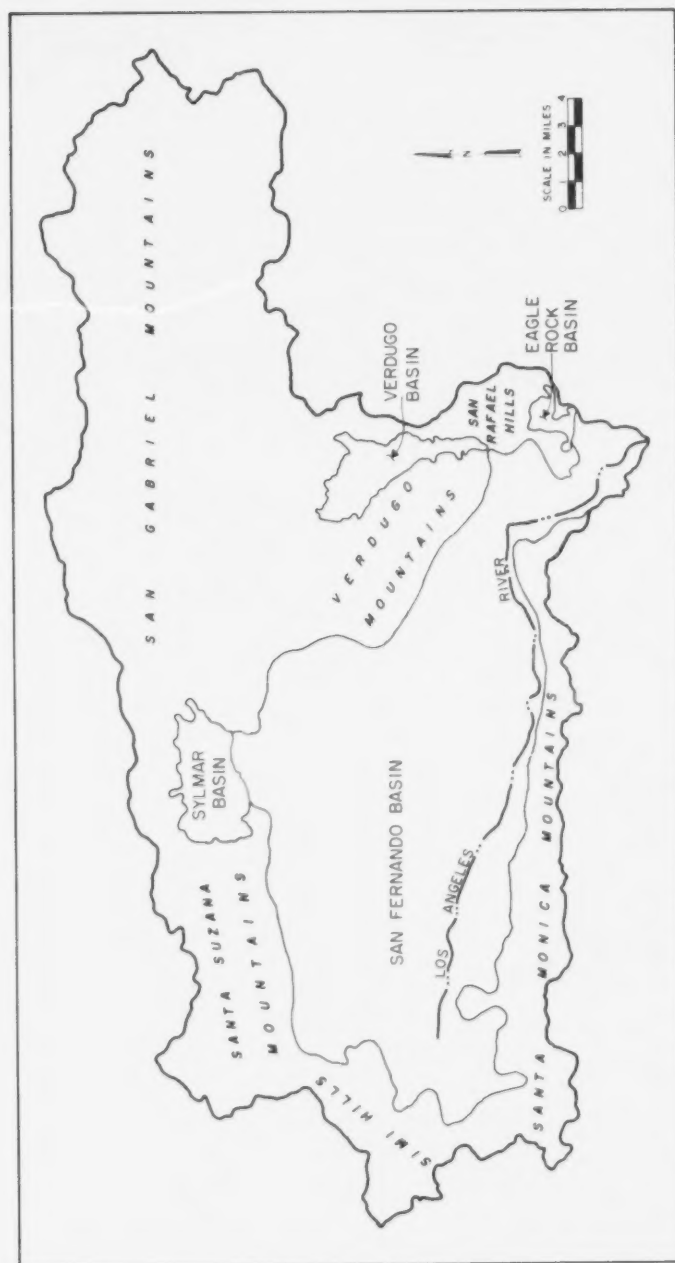


FIGURE 1: UPPER LOS ANGELES RIVER GROUNDWATER BASINS.

FIGURE 2
LOCATION OF SPREADING FACILITIES
IN THE SAN FERNANDO BASIN



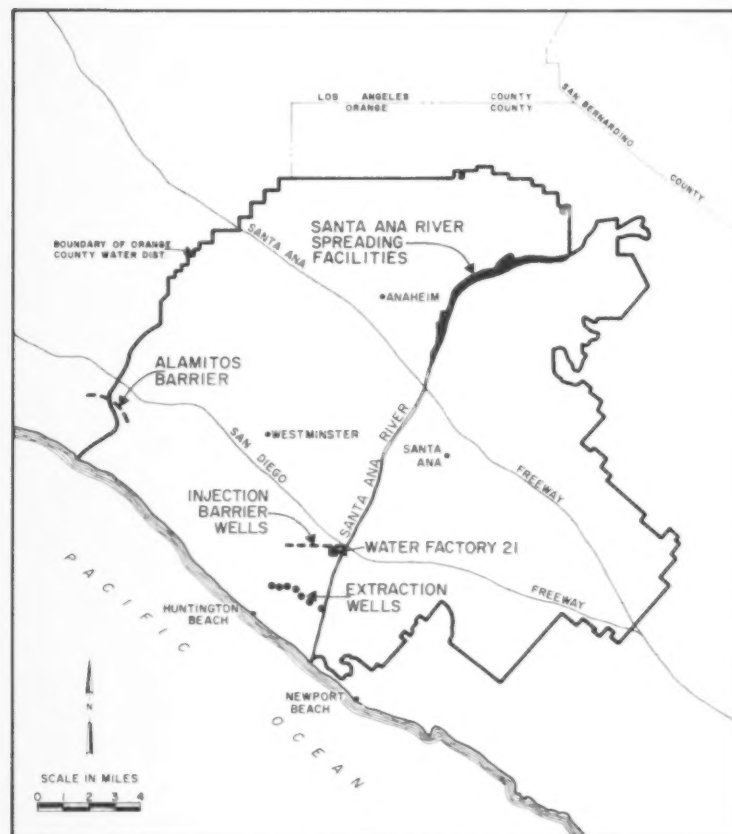


FIGURE 3: ORANGE COUNTY GROUNDWATER RECHARGE FACILITIES AND SEAWATER INTRUSION BARRIERS.

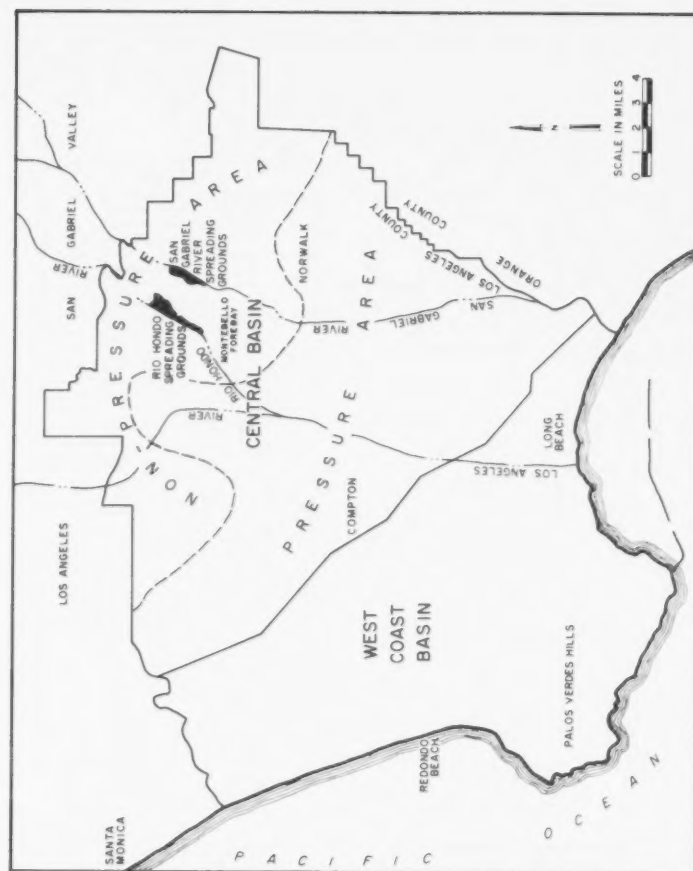


FIGURE 4: GROUNDWATER BASINS WITHIN THE COASTAL PLAIN OF LOS ANGELES COUNTY.

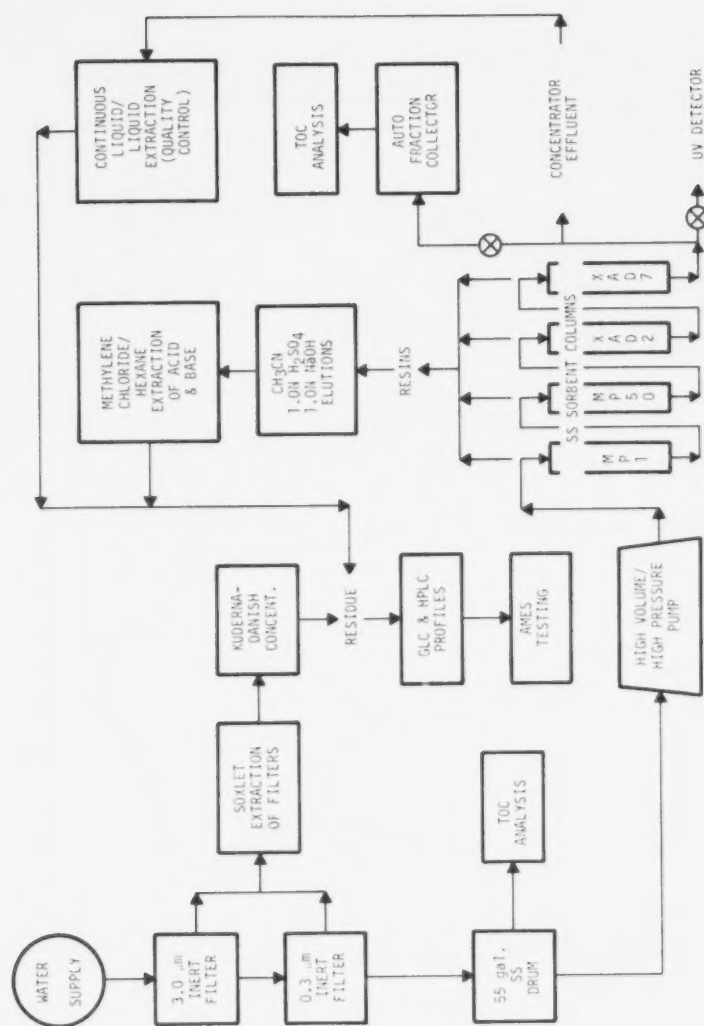
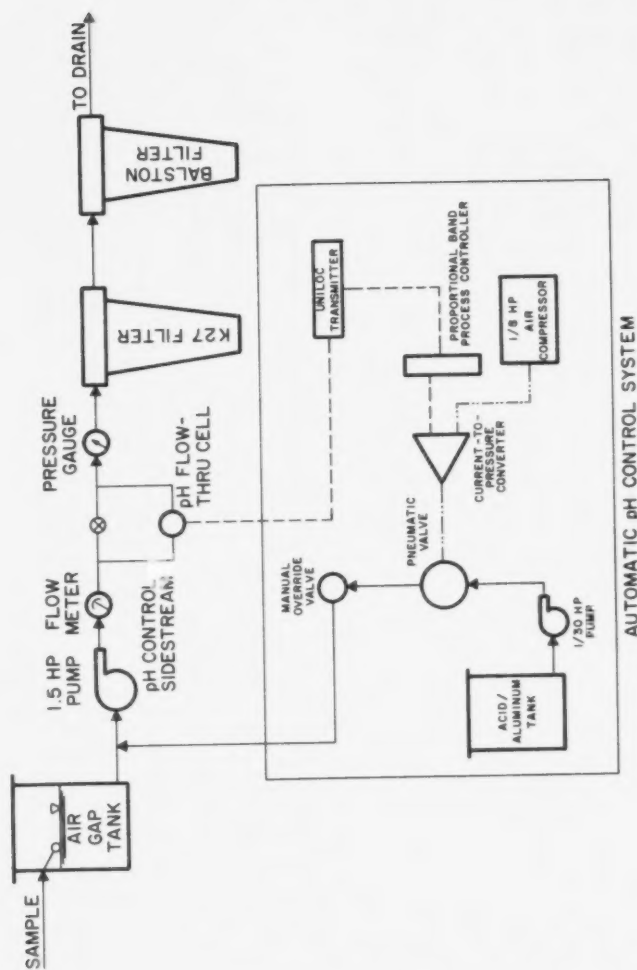


FIGURE 5. HIGH PRESSURE - HIGH VOLUME TRACE ORGANIC CONCENTRATION SYSTEM

FIGURE 6
SCHEMATIC OF VIRUS SAMPLING SYSTEM



AUTOMATIC pH CONTROL SYSTEM

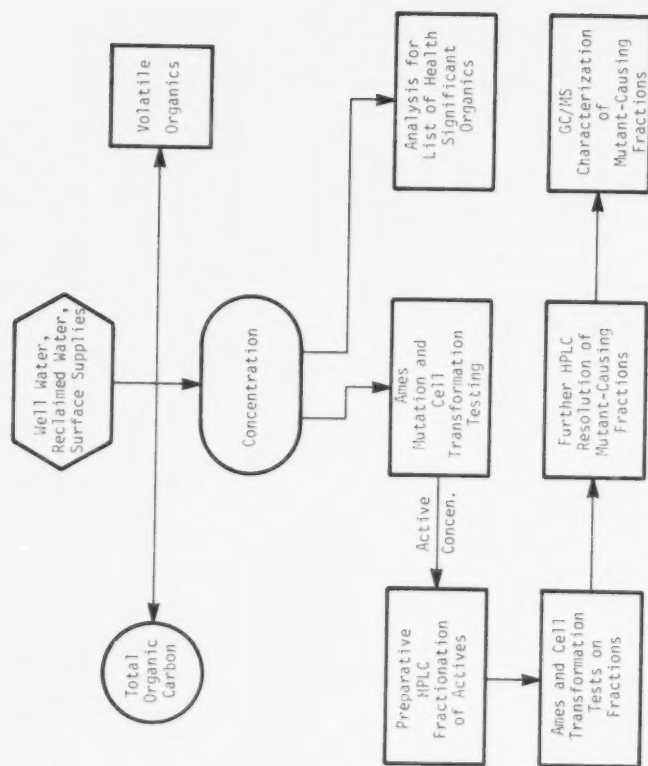


FIGURE 7. ORGANICS/TOXICITY TESTING

FIGURE 8: HEALTH EFFECTS PROJECT SCHEDULE.

MUTAGENIC ACTIVITY AND TRACE ORGANICS IN CONCENTRATES FROM
ADVANCED WASTEWATER TREATMENT PLANT EFFLUENTS

Herbert R. Pahren
Physical Science Administrator
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

Robert G. Melton
Research Chemist
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

There is much interest in certain geographical locations in the potential use of renovated wastewater for potable purpose. These areas may be encountering increased demand for water resources and water planners are looking at various alternatives to supplement existing water supplies.

Since the question of health effects is always raised when consideration is given to potable reuse, the Health Effects Research Laboratory of the U.S. Environmental Protection Agency initiated a project to collect concentrates from advanced wastewater treatment (AWT) plants and to test the concentrates for specific organic compounds and mutagenic activity. AWT plants were chosen since many of the same unit operations will undoubtedly be used if a plant is designed specifically for potable reuse. Data presented in this paper should not be construed as being representative of potable grade water. It is assumed that if research shows the existence of health effects associated with a certain treatment train, the process would be redesigned if potable grade water is to be the objective.

Facilities cooperating in this project are located at South Lake Tahoe, California, Pomona, California, Escondido, California, Orange County, California, Dallas, Texas, and the EPA pilot plant at the District of Columbia. This paper will present the results of mutagenic activity of all samples but the organics data are available for only South Lake Tahoe and Pomona. The organic results from the other plants and an overall assessment will be the subject of a subsequent paper.

DESCRIPTION OF FACILITIES

Details of each AWT plant process will not be presented here since they were previously published in a related report.¹ However, a brief description of the process at the time of sampling will be given.

South Lake Tahoe

Wastewater, at this 7.5 mgd ($0.33 \text{ m}^3/\text{s}$) plant, was treated by primary settling, activated sludge with clarification, high lime coagulation and clarification, ammonia stripping, recarbonation with settling, mixed media filtration, activated carbon, and chlorination.

Pomona

Three separate processes were investigated at 100 gpm (6.3 l/s) flow at Pomona. The initial treatment process for each consisted of primary settling, activated sludge with clarification, and granular activated carbon. Following these steps, the treatment sequences were chlorination alone for system 1, chlorination followed by activated carbon in system 2, and ozonation followed by activated carbon for system 3.

Escondido

Wastewater was first treated in a 208 gpm (13.1 l/s) contact stabilization facility. About 60 percent of the effluent was polished by mixed media filtration and reverse osmosis.

Orange County

A 15 mgd (0.68 m³/s) plant processed wastewater with primary clarification, trickling filtration, high lime coagulation and clarification, ammonia stripping, recarbonation and settling, mixed media filtration, activated carbon, and chlorination.

Dallas

At the 100 gpm (6.3 l/s) facility at Dallas, treatment was by primary settling, trickling filter, secondary settling, activated sludge with clarification, high lime coagulation with clarification, recarbonation, mixed media filtration, activated carbon, and chlorination.

Blue Plains

Two treatment systems were investigated at the 35 gpm (2.2 l/s) Blue Plains pilot plant in Washington, D.C. System 1 consisted of screening, low lime coagulation and clarification, breakpoint chlorination, granular activated carbon, and dual media filtration. System 2 was operated using screening, low lime coagulation and clarification, nitrification, denitrification, activated carbon, dual media filtration, and chlorination.

CONCENTRATION AND EXTRACTION

Concentrates containing the organic materials from the advanced wastewater treatment systems were obtained by Gulf South Research Institute under a contract with EPA. The basic scheme utilized consisted of two recirculating reverse osmosis subsystems in series. The first system utilized a cellulose acetate membrane. Permeate from the cellulose acetate system was reprocessed in a second reverse osmosis system which had a nylon base membrane. Water permeating the nylon membrane was run to waste. Thus, two concentrates were generated: one retained by the cellulose acetate membrane and another retained by the nylon membrane. Between 1,500 and 1,900 liters of plant effluent were processed through the system.

Nylon (polyamide) membranes have been shown to reject many lower molecular weight organics better than those of cellulose acetate. However, polyamide is sensitive to chlorine. The cellulose acetate removes the chlorine, allowing the polyamide to be used.

The aqueous concentrates isolated by reverse osmosis contained a high salt burden in addition to organics. Solvent extraction was used to

avoid the salts as well as concentrate the organics even further. Each concentrate, with volume between 19 and 38 liters, was extracted first with pentane and then with methylene chloride under neutral conditions. Following this, the aqueous phase was adjusted to a pH of 2 and again extracted with methylene chloride. Each of the three extracts was concentrated to one milliliter for testing purposes. Certain modifications of the extraction procedure were incorporated into some of the later samples. Details of the concentration and extraction procedures were outlined in a separate report.¹

MUTAGENIC ACTIVITY

About 80 percent of the concentrate of organics was used to check for mutagenic potential. All of the mutagenic testing was carried out by Stanford Research Institute International under contract to EPA.

Methods

Advanced wastewater treatment plant effluent samples and their solvent blanks were tested by the *Salmonella*/microsome assay procedure described by Ames, et al.² The small amount of sample available necessitated using only *Salmonella typhimurium* strains TA98 and TA100 for most samples. Strain TA98 is sensitive to chemicals that cause frameshift mutations while TA100 is sensitive to frameshift mutagens and base-pair substitution mutagens. Samples generally were tested with and without metabolic activation.

Results and Discussion

Results for the sixty-three samples are presented in Table 1. Most advanced wastewater treatment plants were sampled more than once. In some cases the repeated samples represented different processes, such as Pomona and Blue Plains. In other cases, such as Dallas, Lake Tahoe, and Orange County the same process existed for the sampling which occurred during two different time periods.

Several observations may be made while examining the results of Table 1. Concentrates from the cellulose acetate membrane tended to have more mutagenic potential than concentrates from the nylon membrane. Since the treated wastewaters were passed through the cellulose acetate membrane prior to the nylon membrane, the mutagenic activity possibly may be due to higher molecular weight compounds that did not reach the latter system. The type of solvent extraction would determine the type of organics which would be contained in a given fraction of concentrate and hence whether one fraction may be expected to have greater adverse effects than another. It may be noted that methylene chloride extraction under acidic conditions has a greater number of positive responses for mutagenicity than with pentane or neutral methylene chloride. Although it may or may not be significant, 6 out of 9 systems which had chlorination as the final unit operation showed mutagenic properties in the concentrate. The systems with either activated carbon or reverse osmosis as the final step were all negative. For a substance to be present in the sample, it must be rejected by the reverse osmosis membrane used in the concentration system. Since reverse osmosis was used to treat the wastewater at Escondido, compounds not capable of permeating a membrane most likely were removed.

TABLE 1
Mutagenic Potential of Advanced Wastewater Treatment Effluents

AWT Plant	Membrane*	Solvent**	Mutagenic
Dallas I	CA	P	No
	CA	MC-neutral	No
	CA	MC-acidic	Yes
	N	P	No
	N	MC-neutral	No
	N	MC-acidic	No
Dallas II	CA	P	No
	CA	MC-neutral	No
	CA	MC-acidic	Yes
	N	P	No
	N	MC-neutral	No
	N	MC-acidic	Slight
Lake Tahoe I	CA	P	No
	CA	MC-neutral	No
	CA	MC-acidic	Yes
	N	P	Yes
	N	MC-neutral	Yes
	N	MC-acidic	Yes
Lake Tahoe II	CA	P	No
	CA	MC-neutral	Slight
	CA	MC-acidic	Yes
	N	P	No
	N	MC-neutral	No
	N	MC-acidic	Slight
Pomona I	CA	P	Yes
	CA	MC-neutral	Yes
	CA	MC-acidic	Yes
	N	P	No
	N	MC-neutral	No
	N	MC-acidic	No
Pomona II	CA	P	No
	CA	MC-neutral	No
	CA	MC-acidic	No
	N	P	No
	N	MC-neutral	No
	N	MC-acidic	No
Pomona III	CA	P	No
	CA	MC-neutral	No
	CA	MC-acidic	No
	N	P	No
	N	MC-neutral	No
	N	MC-acidic	No

TABLE 1
(Continued)

Mutagenic Potential of Advanced Wastewater Treatment Effluents

AWT Plant	Membrane*	Solvent**	Mutagenic
Escondido	CA	P	No
	CA	MC-neutral	No
	CA	MC-acidic	No
	N	P	No
	N	MC-neutral	No
	N	MC-acidic	No
Blue Plains I	CA	P	No
	CA	MC-neutral	Yes
	CA	MC-acidic	Yes
	N	P	No
	N	MC-neutral	No
	N	MC-acidic	No
Blue Plains II	CA	P	No
	CA	MC-neutral	Slight
	CA	MC-acidic	No
	N	P	Slight
	N	MC-neutral	No
	N	MC-acidic	No
Composite			Slight
Orange County I			
Composite			No
Orange County II			
Composite			No

*CA (Cellulose acetate); N (Nylon)

**P (Pentane); MC (Methylene chloride)

ORGANICS

About 20 percent of most concentrates obtained by Gulf South Research Institute is being utilized to determine specific organic compounds in the samples. In addition to attempting to identify the compounds in greatest concentration, the contractor performing the analytical work, Battelle Columbus Laboratories, is searching for and quantifying 54 organic compounds of special interest to the EPA Health Effects Research Laboratory. These special interest compounds are listed in Table 2. Separate samples were taken from the reverse osmosis feedstock for volatile organic analyses but the results are not considered valid because of routine shipping delays, so these data are not presented. The organic procedure used will determine compounds generally within the range of molecular weight from 70 to 500.

Methods for Chemical Analysis

Basically, the organic concentrates are analyzed by addition of deuterated internal standards to the concentrate, fractionation of the concentrate into acidic, basic, and neutral fractions, partitioning the neutral fraction on a micro-silica gel column, and analysis of the unpartitioned concentrate and partitioned fractions by glass capillary gas chromatography using a flame ionization detector (GC-FID) and glass capillary gas chromatography coupled to a mass spectrometer (GC/MS). The GC-FID and GC/MS analyses are conducted on two types of wall-coated open-tubular glass capillary GC columns, one containing a nonpolar liquid phase (SP2100) and the other a polar liquid phase (SP1000). The basic analytical approach was developed at the EPA Health Effects Research Laboratory. This approach requires extensive quality assurance procedures in order to permit meaningful comparison of concentrates over the 3-year period of the project. Details of the chemical analysis procedures have been previously described^{4,5} and are summarized in Figure 1.

Partitioning of the Concentrates. The concentrates, prior to chemical analysis, are complex mixtures of organics dissolved in either pentane or methylene chloride. Fourteen deuterium-labeled compounds are added to each concentrate before fractionation of the concentrate is begun.⁶ Also, hexaethylbenzene is added to each fraction just before GC-FID and GC/MS analysis. These internal standards perform the following purposes: (1) monitoring fractionation recoveries, (2) calculation of relative retention times, and (3) quantitation of compounds identified in the concentrate and partitioned fractions.

The partitioning scheme is described in Figure 2. This scheme is a modification of the procedure reported by Ishiwatari and Hanya.⁷ The acidic fraction is derivatized with diazomethane to convert the organic acids and phenols into methyl esters and ethers. The neutral fraction is further partitioned using a modification of a standard method for the separation of pesticides.^{8,9} This method uses open column chromatography on 5% deactivated silica gel packed in a 6-inch disposable Pasteur pipette. After partitioning, an aliquot of each concentrate fraction and blank fraction is evaporated to constant weight and weighed.

Gas Chromatographic Separations. The partitioned fractions are still very complex, so high resolution glass capillary columns are used to further separate the mixtures. Each unpartitioned concentrate and fraction is analyzed on a polar (SP1000) and non-polar (SP2100) capillary column. Organics which co-elute on one column are usually separated on the other column. Each column is extensively tested to meet minimum performance requirements on a daily basis.

TABLE 2
Special Interest Compounds

MW	Compound Name
(1) 88	Chloroprene
(2) 93	Aniline
(3) 94	Phenol
(4) 102	Diethylnitrosamine
(5) 104	Styrene
(6) 108	p-Methyl phenol
(7) 118	2-Methyl styrene
(8) 122	2,4-Diaminotoluene
(9) 122	2,4-Dimethyl phenol
(10) 123	Nitrobenzene
(11) 126	2-Chlorotoluene
(12) 127	2-Chloroaniline
(13) 128	4-Chlorophenol
(14) 138	3-Nitroaniline
(15) 142	4-Chlorophenol me-ether
(16) 143	2-Naphthylamine
(17) 146	1,4-Dichlorobenzene
(18) 153	p-Nitrophenol me-ether
(19) 154	4-Chlorophenyl methyl ketone
(20) 162	2-Chloronaphthalene
(21) 169	Diphenylamine
(22) 169	4-Phenylaniline
(23) 176	2,4-Dichlorophenol me-ether
(24) 180	1,2,4-Trichlorobenzene
(25) 196	2,4-Dichloronaphthalene
(26) 202	Fluoranthene
(27) 210	2,4,6-Trichlorophenol me-ether
(28) 214	Methylchlorophenoxy acetic acid me-ester
(29) 219	n-Phenyl-1-naphthylamine
(30) 228	Chrysene
(31) 234	2,4-Dichlorophenoxy acetic acid me-ester
(32) 234	Dibromobenzene
(33) 252	3,4-Benzpyrene
(34) 252	Benzfluoranthene (b,k)
(35) 258	Hexachloro-1,3-butadiene
(36) 263	Pentachloroaniline
(37) 268	2,4,5-Trichlorophenoxy acetic acid me-ester
(38) 270	Hexachlorocyclopentadiene
(39) 276	Benz-(ghi)-perylene
(40) 276	Indeno-(1,2,3-cd)-pyrene
(41) 278	Pentachlorophenol me-ether
(42) 282	Hexachlorobenzene
(43) 288	Lindane
(44) 290	Tetrachlorobiphenyl
(45) 293	Pentachloronitrobenzene
(46) 324	Pentachlorobiphenyl
(47) 326	Triphenyl phosphate
(48) 352	DDT
(49) 358	Hexachlorobiphenyl
(50) 362	Aldrin
(51) 368	Tri-(m)-cresyl phosphate
(52) 370	Heptachlor
(53) 378	Dieldrin
(54) 622	Hexabromobiphenyl

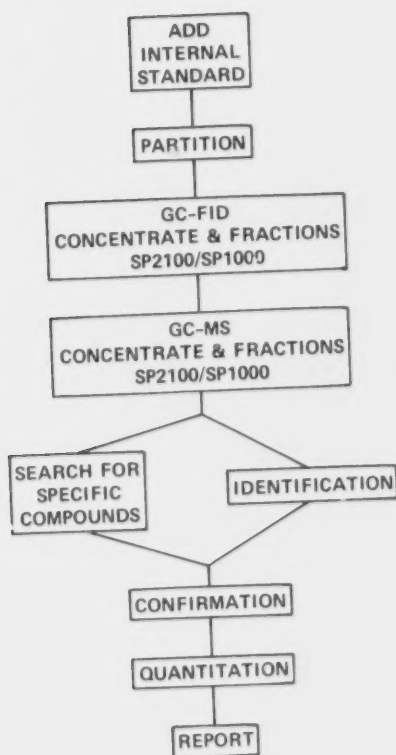


Fig. 1. Analytical Scheme for Analysis of Water Concentrates



Fig. 2. Partitioning Scheme for Fraction of Water Concentrates

All partitioned fractions and their corresponding blank samples and unpartitioned concentrates are analyzed by GC-FID on both capillary columns prior to analysis by GC/MS. The chromatograms resulting from these GC-FID analyses are retained for comparison with the total-ion-current chromatograms generated by the subsequent GC/MS analyses. Any abnormal differences between the GC-FID and GC/MS chromatograms indicate a chromatographic or instrumental problem.

Gas Chromatographic-Mass Spectrometric Analyses. GC/MS analysis of the partitioned fractions and the unpartitioned concentrate, as well as their corresponding blank samples, provides the primary basis for identification and quantitation of organic compounds in the concentrate. In order to ensure that all GC/MS data acquired on this project are of high quality, the total GC/MS system performance is rigorously evaluated each day before undertaking analysis of concentrate samples.¹⁰

A total of 18 GC/MS data files are acquired per concentrate. They include the data on the unpartitioned concentrate and four fraction/blank pairs acquired on both the SP1000 and SP2100 capillary GC columns.

Organics are initially identified based on a computer or manual comparison of the unknown mass spectrum with a library of reference mass spectra. The Finnigan/Incos 2300 data system mass spectral matching computer programs were used for computer identifications because they have recently been shown to be most effective.¹¹ The data base for the computer matching is a library of 31,357 spectra from the National Bureau of Standards plus additional spectra acquired by Battelle Columbus Laboratories. The 54 compounds (Table 2) considered to present significant health risks are specifically searched in the MS data files using a computer program that utilizes selected ion summation and specific retention time windows.¹² Compound identification is considered tentative until it can be confirmed by comparison of its mass spectrum and relative retention time with the corresponding data obtained on an authentic reference sample under the same GC/MS conditions. Compounds which are found in the blanks in approximately the same amount as found in the samples are assumed to result from sample processing and are not reported as being present in the original concentrate.

Each of the 54 compounds in Table 2 are quantitated using a previously described procedure.⁴ The calculations assumed 100 percent recovery through the concentration and analytical steps, which is probably not correct. It is much too expensive to quantitate every organic compound identified in every sample; therefore, a second, less precise method of estimating the relative amount of each organic was devised.⁴ This method assigns a relative size (RS) value by visually estimating the size of each peak in the total ion chromatogram relative to that of hexaethylbenzene, the internal standard. Thus, RS values are directly comparable from one concentrate report to another.

Quality assurance measures for the 3 year period of the chemical analysis of these AWT concentrates are extensive and have been previously described.⁴

Data Management. In order to provide meaningful comparisons of the chemical data on the various concentrates, a comprehensive computer data management system was implemented. All data relative to the chemical characterization of identified compounds are entered into a large, off-line computer located at Battelle Columbus Laboratories. These data include the following items:

1. CAS number
2. Common name
3. Systematic name
4. Industrial source and/or use
5. Molecular weight
6. Molecular formula
7. Retention time
8. Spectrum number
9. Relative size
10. Compound classification
11. Functional group classification
12. The fraction in which it is identified
13. The column used
14. Quantitation data

In addition to the above items relating to each identified compound, the following information is also entered into the master data base:

1. Descriptive data on the concentrate
2. Gravimetric data
3. Conditions used for the GC/MS analyses

Mass spectra of concentrate compounds which cannot be identified are catalogued according to their retention times. The computer data handling system has the capability to edit and update the data base of any concentrate at any time.

Results and Discussion

As indicated previously, the organics portion of the overall project has not been completed. Results are available at this time only for the Lake Tahoe II effluent concentrated by the cellulose acetate (CA) membrane, the three CA concentrates from Pomona I, the CA concentrate from Pomona II using acidified methylene chloride extraction, and a composite of the three CA extracts from Pomona III. It is not intended to determine the organic compounds in each sample checked for mutagenic activity, as the cost would be prohibitive. Rather, a representative number of samples have been selected from the various sites so as to include most concentrates showing mutagenic activity as well as several which were free of mutagenic activity. In several cases, as with Pomona III, the concentrates from the several fractions were composited.

A total of 447 compounds were identified in the Lake Tahoe II samples. Using the semi-quantitative procedure of relative size to estimate quantities, the compounds in highest concentration were clofibrac acid at 2 ppb, followed by di-n-butylphthalate and tripropylene glycol, methyl ether. Most of the compounds were at the parts per trillion level. It is interesting that clofibrac acid is the compound in greatest concentration. Clofibrate is a drug used to reduce lipids in the blood stream and a patient is usually given 2 grams per day for this purpose. A person on clofibrates will excrete clofibrac acid in the urine. The procedure detected 4 mg in the 400 gallon sample processed.

A specific search was made for the 54 special interest compounds listed in Table 2 and ten were detected. Reference samples for each of the ten were used to quantitate the compounds and the results are presented in Table 3. Pentachlorophenol, a wood preservative, was present in highest concentration at 16.58 ppt. Comparing the results of the mutagenic activity in Table 1, which showed that extract of methylene chloride

TABLE 3

Special Interest Compounds in Lake Tahoe II Effluent

Compound	Concentration in ng/l		
	Pentane	Meth C1 Neutral	Meth C1 Acidic
Phenol		1.09	8.52
Diphenylamine	1.54	0.03	
Styrene	0.86	1.03	0.36
o-Chloroaniline	0.63		
Fluoranthene	0.20		
p-Chlorophenol		1.56	8.06
2-Methyl styrene		0.05	0.61
Hexachlorobenzene			0.14
Pentachlorophenol	16.58		
Chrysene	5.15		

TABLE 4

Occurrence of Functional Groups in Lake Tahoe II Extracts

Compound	Number Identified		
	Pentane	Meth C1 Neutral	Meth C1 Acidic
Unsubstituted aliphatic	3	3	17
Halogenated aliphatic	5	4	1
Substituted aliphatic	48	61	48
Unsubstituted alicyclic	9	0	8
Halogenated alicyclic	0	0	0
Substituted alicyclic	17	22	14
Unsubstituted aromatic	145	8	31
Halogenated aromatic	4	2	5
Substituted aromatic	38	35	45
Unsubstituted PNA	92	5	19
Halogenated PNA	0	0	0
Substituted PNA	6	4	5
Alcohol group	32	27	26
Aldehyde group	3	2	2
Amide group	1	0	2
Amine group	4	1	3
Carboxylic acid	20	40	42
Ester group	32	22	9
Ether group	14	19	18
Heteroatom in ring	10	3	10
Ketone	13	21	12
Nitro group	3	10	2
Phthalic di-ester	4	3	2
Phenol group	10	3	12
Sulfur atom	1	1	4
Totals	514	296	337

under acidic conditions was mutagenic while the pentane extract was not, to the compounds of health significance in Table 3, does not provide any reason for the differences of mutagenic activity.

A listing was made of the functional groups associated with the concentrates from Lake Tahoe II and this listing is shown in Table 4. The pentane extract, which was not mutagenic, had a greater total number of functional groups than the other two extracts that showed signs of mutagenic activity, but there was no indication of the possible cause of the mutagenic activity. Similar tables from other samples are needed before one could check whether or not a pattern exists.

A combined listing of compounds for all three extracts from system 1 at Pomona shows 389 identified compounds. Of these, 194 organic compounds were identified in the acidified methylene chloride extract alone. This compares to 119 compounds in the acidified methylene chloride extract of system 2 and 108 compounds in the composite of all six extracts from system 3. Thus, it appears that ozonation reduces the number of organic compounds and the two systems having activated carbon as the final treatment step results in fewer compounds than with system 1 where chlorination was the final step.

Compounds in highest concentrations in the Pomona samples, as determined by the semi-quantitative relative size procedure, include dioctyl sebacate -1.5 ppb, di-n-butylphthalate - 1.5 ppb, and tris-B-chloroethyl phosphate -1 ppb.

Special interest compounds, with their concentrations, are listed in Table 5 for each of the five Pomona samples analyzed. The compound in highest concentration was 2,4-D, a weed killer. Following the trend for the total number of compounds, systems 2 and 3 had fewer numbers of special interest compounds than with system 1, and the concentrations tend to be lower. When comparing the Table 5 results to Table 1, where it was shown that all three samples obtained by the cellulose acetate membrane in Pomona system 1 had mutagenic activity while no mutagenic activity was observed in the samples from systems 2 and 3, only a gross observation may be made that the system with higher organics had mutagenic activity.

Functional groups for the Pomona compounds are compared in Table 6. The totals exceed the total number of compounds identified because some compounds have more than one functional group. Comparison of the Pomona I results which had mutagenic activity with functional groups for Pomona II and III which showed no mutagenic activity does not seem to show any pattern related to such groups.

As more data become available regarding the analysis of organic concentrates, perhaps patterns will become more pronounced. On the other hand, the possibility is very strong that positive results with the Ames test cannot be related to identified organics or functional groups in a sample. When dealing with complex mixtures of organics at low levels, biological screening tests do not have the same sensitivity as available with chemical testing procedures. Perhaps only when new or improved biological testing systems are available will we be able to say with assurance whether or not exposure to these materials will result in any adverse health effects.

TABLE 5
Special Interest Compounds in Pomona Effluent

Compound	Concentration in ng/l				
	Pomona I Pentane	Pomona I Meth Cl Neutral	Pomona I Meth Cl Acidic	Pomona II Meth Cl Acidic	Pomona III Composite
Phenol	4.85	19.68	11.26	4.56	3.92
Styrene	0.84	1.41	0.61		0.84
2-Methyl styrene	0.59	0.64	0.28	0.09	0.11
2-Chloroaniline	3.86	24.87			4.52
Diphenylamine	9.77	2.18	0.14		0.05
2,4,6-Trichlorophenol	1.59		2.42	1.31	9.64
1,2,4-Trichlorobenzene	0.70		0.08		
Fluoranthene	3.43	0.22			
Pentachlorophenol	43.92	10.17	6.77	11.06	
Triphenyl phosphate	1.31				
p-Chlorophenol		16.41	54.82		
2,4-D			69.68		
Hexachlorobenzene			3.40	1.55	
p-Chloroacetophenone		7.49			
Tricresylphosphate					2.6
p-Dichlorobenzene			0.56		0.02

TABLE 6
Occurrence of Functional Groups in Pomona Concentrates

Group	Number Identified				
	Pomona I Pentane	Pomona I Meth Cl Neutral	Pomona I Meth Cl Acidic	Pomona II Meth Cl Acidic	Pomona III Composite
Unsubstituted aliphatic	1	3	6	3	6
Halogenated aliphatic	1	3	7	3	1
Substituted aliphatic	43	55	58	52	25
Unsubstituted alicyclic	5	0	5	0	0
Halogenated alicyclic	0	0	2	0	0
Substituted alicyclic	4	6	17	15	2
Unsubstituted aromatic	67	28	38	9	41
Halogenated aromatic	10	9	17	4	10
Substituted aromatic	35	46	48	31	38
Unsubstituted PNA	49	21	23	7	26
Halogenated PNA	0	0	1	0	1
Substituted PNA	4	6	4	2	2
Alcohol group	24	25	31	24	13
Aldehyde group	1	3	4	2	0
Amide group	2	4	4	5	1
Amine group	6	8	9	6	4
Carboxylic acid	13	43	45	41	15
Ester group	20	14	4	15	13
Ether group	11	22	32	13	10
Heteroatom in ring	8	20	23	14	2
Ketone	8	19	31	7	3
Nitro group	1	4	2	1	1
Phthalic di-ester	3	2	2	4	10
Phenol group	15	11	8	4	5
Sulfur atom	2	5	5	4	2
Totals	333	357	426	266	231

We cannot rule out the possibility that certain compounds, such as plasticizers, are added to the concentrates during the concentration process. Studies are underway at present to try to identify the degree of such sample contamination. There are sufficient differences, however, in the mutagenesis and chemical data from site to site for given fractions to conclude that most compounds must have been in the original wastewater and are not an artifact of the concentration process.

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BIOLOGICAL EVALUATION OF TOXIC EFFECTS OF ORGANIC
CONTAMINANTS IN CONCENTRATED RECYCLED WATER

Nachman Gruener
Associate Professor
Environmental Health Sciences
Tulane University
School of Public Health & Tropical Medicine
New Orleans, Louisiana 70112

Water used to be one of our cheapest natural resources. In recent years, we have discovered that water is no longer an abundant commodity in many places in the United States and other countries of the world. Reasons for this include population growth, increases in the average water consumption for domestic and industrial purposes, pollution, and climatic changes. Although there are annual changes in our water reservoirs, the long-term trends show that shortage of water is going to be a problem in many places that have not experienced this phenomenon before.

One of the approaches to remedy this situation is conservation. Natural reuse of water or unintentional reuse by humans has been known for many years. But the presence of thousands of compounds in our wastewater, among them known toxins and carcinogens, should alarm us to take a more cautious approach when we consider wide-scale usage of treated wastewater. Treated wastewater can be used for a variety of purposes - agriculture, industry, recreation, drinking and other nonpotable applications. Treated wastewater may be used for drinking water in several ways, such as recharge of ground water or direct supply with or without other sources of water. None of these strategic approaches have been dealt with in a practical way. Obviously, water should get special toxicological consideration since it is a general daily commodity used throughout our life and by everyone in the population.

For the purpose of the evaluation of the health effects of long-term consumption of reused water, an extensive toxicological study was planned. A preliminary problem was sample preparation. Because of the inherent weakness of a toxicological experiment with regard to extrapolation to humans, levels at least 100 times the human daily consumption had to be evaluated. The water, therefore, had to be concentrated. Many concentration systems are inappropriate because the end product is selective both in relation to removal of original compounds or addition of contaminants. Preliminary laboratory studies showed that the best way to concentrate the water was through reverse osmosis technology. Since the study was intended to test only the organic soluble fraction, we decided to use charged membranes to remove much of the inorganic fraction. This was necessary for toxicological and nutritional reasons. Some 400,000 liters of advanced treated wastewater was concentrated down to 200 liters through several steps. The first step was done in the treatment plant and took almost 2 months. The first concentrate (757 liters) was brought to the laboratory where the additional steps were carried out.

Table I shows the various stages of sample preparation with the total organic carbon (TOC, in short) recovery values. The concentrated water was incorporated into a gel-type diet which was balanced with regard to the inorganic ions based on the concentrations found in the concentrated sample. Chemical analysis of the concentrate is shown in Table II.

As we can see, all of the toxic heavy metals were present at low concentrations or could not be detected. The project included 900 mice of the strain CFl, males and females. There were five groups in each study: controls and four others exposed to 45, 90, 180 and 360 milligrams TOC per kilogram of food. The intake of the TOC in the different groups was in the range of 5-150 mg TOC/kg B.W./day, according to the exposure level, age and sex. These levels are 25-2500 times the levels to which humans are normally exposed. The different studies that were performed with the mice are shown in Table III.

The toxicological tests that were carried out are exhibited in Table IV.

The mice in the preliminary 14-day experiment were checked for body weight, food consumption and general physiology. They also were tested for mutagenicity by the dominant lethal mutation test. In Study I, the mice were tested by all the above tests and also for hematology, blood chemistry, mixed function oxidase activity, and motor activity. Similar tests were performed in Studies II, III, and IV.

The results of the different tests which were run during the various studies showed that there were no significant differences in the hematological determinations, motor activity measurements, mixed function oxidase activities or body weight values. The reproduction tests did not show any physiological deviations. In the blood chemistry determinations, which include 20 tests per group, there were small numbers of parameters which were found to be significant (P is lower than 0.05).

When dealing with a large number of variables and a rather large number of animals, one can expect that there will be significant differences between experimental and control values for some of the parameters. For this reason, we insist that experimentally significant results will be only those which were found in more than one experimental group.

Males in Study I showed higher levels of sodium, chloride and CO_2 in the serum of experimental groups. Males in Study II had lower urea values in the experimental groups and higher levels of serum proteins. Males in Study III had lower glucose values, lower alkaline phosphatase activity and lower triglycerides. In the females of this study, cholesterol showed lower values in the experimental groups. As a whole, the blood chemistry did not indicate a specific or general pathological situation in these animals. Among the tissues, the spleen mean weight was higher in males of Studies II and III, and in the females of Study III. The adrenals were bigger in males and females of Study III.

The evaluation of the mutagenic potential of this water was done in *in vivo* and *in vitro* experiments. Table V shows the first dominant lethal mutation test done with the mice.

The mean of the dead fetuses was found to be significantly higher than the control. In a second experiment, in spite of the fact that the mean of the dead fetuses was even higher, the difference between the experimental and the control group was found to be insignificant (Table VI).

To try to resolve this discrepancy, we tried other mutagenicity tests: a microbiological test using the *Salmonella* strains, and the mammalian mutagenicity assay. The microbiological tests did not show any mutagenicity. The experiment with mammalian cultures was done with V79 hamster cells. In this test, the cells were checked for the presence of forward mutation, in our case, ouabain-resistant cells. The results presented in Table VII shows that at the dose of 200 μg of TOC (0.3 ml

of water), the mutagenicity rate is 20 times the control.

In vitro toxicity tests were run with human lung fibroblasts (W138). Figure 1 shows the direct effect of the water on the protein level in the cell culture.

It can be seen that toxicity increases with time. Figure 2 shows that the presence of the liver activation system significantly increases the toxicity.

Both experiments were done in the presence of 150 µg TOC.

In summary, concentrated recycled water with TOC levels up to about one thousand times the expected human exposure did not cause pathological changes in the various toxicological tests done in mice after exposure for up to five months. On the other hand, the concentrated water was found to be mutagenic. More studies on the mutagenicity and carcinogenicity of renovated water are necessary before conclusions can be made.

Acknowledgement

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TABLE I. ORGANIC LEVELS AT VARIOUS STAGES OF SAMPLE PREPARATION

Process Step	Sample Volume (liters)	Sample TOC (mg/l)	Mass of Organics (g)	Total Organic Mass Retention (%)	Step-Wise Organic Mass Retention (%)
Initial reverse osmosis concentration	757	530	402.1	53.0	53.0
Alkaline precipitation/filtration	757	384	290.7	38.3	72.5
Closed-loop dialysis	757	260	196.8	26.0	67.7
Final reverse osmosis concentration	204	907	185.5	24.5	94.3
SO ₄ ²⁻ precipitation/filtration	204	704	144	19.0	77.6

TABLE II. CHEMICAL ANALYSIS OF THE CONCENTRATE

Parameter	mg/l	Parameter	mg/l
As	<0.01	Na	1300
Ba	<0.5	K	4300
Cu	<0.10	Ca	632
Al	0.40	Mg	15
Cr	<0.01	Cl	4400
Zn	2.60	SO ₄	2650
Pb	<0.01	PO ₄	1500
Ag	<0.01	I	2.9
Hg	<0.01	Total Hardness	1640
Se	<0.01	(as CaCO ₃)	
Mn	0.17	Mg Hardness	60
Fe	1.70	(as CaCO ₃)	
Co	<0.20	Total Alkalinity	270
B	<15	Total	132,000
Si	1.3	Dissolved Solids	
NO ₃ -N	<0.1	pH	5.5
NO ₃ -N	<0.01	Stability index	7.3
F	2.6	Saturating index	0.9
		Odor Threshold	4
		Turbidity	3.2
		Color PCU	260
		Total Organic	704
		Carbon	

TABLE III. SUMMARY OF STUDIES PERFORMED

Study	Exposure Time	NUMBER OF ANIMALS	
		Males	Females
P	14 days	50	100
I	90 days	200	200
II	Gestation, Lactation and another 90 days	100	100
III	Gestation, Lactation and another 150 days	50	50
IV	90 days and another 90 days on regular diet	50	-

TABLE IV. TOXICOLOGICAL TESTS (IN VIVO)

General Physiology	Blood Chemistry
Food Consumption	Glucose
Body Weight	Cholesterol
Hematology	Triglyceride
Hemoglobin	Total Protein
Red Blood Cell Count	Albumin
White Blood Cell Count	Calcium
Mixed Function Oxidase Activity	Phosphorus
Motor Activity	Sodium
Reproduction	Potassium
Dominant Lethal Mutation	Chloride
Pathology	Carbon Dioxide
Heart	Urea Nitrogen
Lungs	Uric Acid
Spleen	Total Bilirubin
Liver	Creatinine
Kidney	Alkaline Phosphate
Adrenals	LDH
Brain	GOT
Tests	GPT
Ovaries	CPK
Microscopy and Tissue Weights	

TABLE V. EFFECTS OF ADMINISTRATION OF CONCENTRATED RENOVATED WATER ON SURVIVAL OF FETUSES
(Study I)

Group	Number of Females		Percentage of Pregnancy	Number of Fetuses		Percent of Fetuses		Mean Number of Fetuses per Pregnant Female	
	Pregnant	Non-Pregnant		Total	Live	Dead	Live	Total	Live
A	84	16	84	926	889	37	96.0	11.0	0.4
B	81	18	82	913	853	60	93.4	11.3	0.8
C	76	22	78	891	830	61	93.2	11.7	0.8
D	63	35	64	686	633	53	92.2	10.9	0.9
E	65	24	73	678	632	46	93.2	10.4	0.7

TABLE VI. EFFECT OF ADMINISTRATION OF CONCENTRATED RENOVATED WATER ON SURVIVAL OF FETUSES
(Study II)

Group	Number of Females		Percentage of Pregnancy	Number of Fetuses		Percent of Fetuses		Mean Number of Fetuses per Pregnant Female	
	Pregnant	Non-Pregnant		Total	Live	Dead	Live	Total	Live
A	71	26	73	855	797	58	93.2	12.0	0.8
B	72	25	74	903	830	73	91.9	12.5	1.0
C	81	19	81	988	906	82	91.7	12.2	1.0
D	80	20	80	997	918	79	92.0	12.5	1.0
E	76	24	76	993	848	85	90.9	12.3	1.1

TABLE VII. MUTAGENICITY OF CONCENTRATED RECYCLED WATER (first run)^a

Sample	Toxicity (% Plating Efficiency)	Colonies per 16 Plates	Oubain-Resistant Mutants per 10 ⁶ Survivors
Control	88	1	0.5
Control + S9	78	0	0.0
0.1 ml H ₂ O+S9	44	1	1.0
0.2 ml H ₂ O+S9	46	4	3.8
0.3 ml H ₂ O+S9	24	6	10.9
0.5 µg BP+S9	55	54	43.8
1.0 µg BP+S9	38	104	142.2

^aH₂O mixed 1:1 with 2 x MEM and added to each sample with 1 ml S9 mix plus PBS to give 2 ml final volume. TOC level in water was 700 µg/ml. Cells incubated with S9 alone have a plating efficiency of 70%. S9 is the rat liver homogenate supernate after centrifugation at 9000 g. BP-Benzo(a)pyrene.

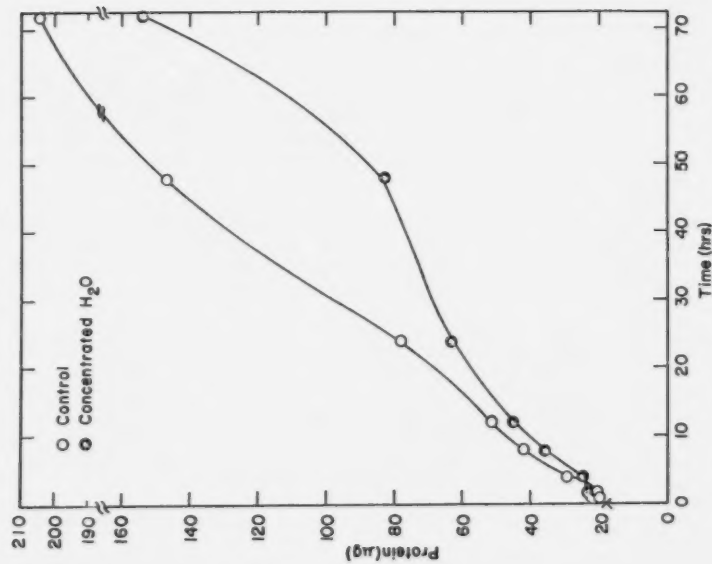


Figure 1. Direct exposure of W138 to concentrated recycled water.

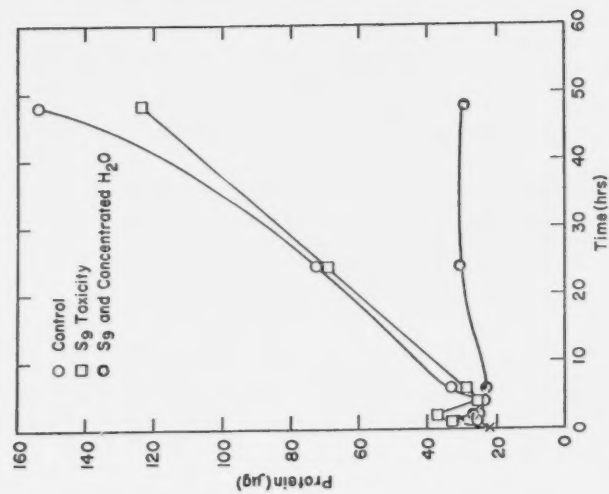


Figure 2. Effect of liver activation system on toxicity of concentrated recycled water in W138 cells.

DIE-AWAY KINETICS OF AEROSOLIZED BACTERIA
FROM SPRINKLER APPLICATION OF WASTEWATER*

Benjamin Tetsch
Research Assistant

Hillel Shuval
Professor of Environmental Health

Jacob Tadmor
Visiting Professor of Environmental Sciences
Division of Human Environmental Sciences
School of Applied Science and Technology
The Hebrew University
Jerusalem, Israel

INTRODUCTION

The need to predict the concentration of aerosolized pathogenic microorganisms downwind from a wastewater spray or aeration site has grown in importance with the renewed impetus given to wastewater reuse and disposal by land treatment.

In many of these applications, wastewater effluent is sprayed under pressure with 0.1 to 1 percent of the liquid being aerosolized (7). In wastewater treatment processes involving aeration, aerosols are formed containing concentrations of enteric microorganisms 1000 times or more greater than that detected in the ambient liquid. In a similar manner, enteric microorganisms from wastewater disposed of into the sea are aerosolized at significantly higher concentrations than found in the source water, and can be carried air-borne to nearby shore areas.

Air monitoring of such aerosolized microorganisms has led to the detection of enteric microorganisms as much as 1200 m downwind from a sewage treatment plant and 350 m downwind from a sewage sprinkler irrigation field. Similarly, pathogenic enteric salmonella bacteria and viruses have been detected 40 - 100 m downwind from sewage spray irrigation nozzles (11).

The potential health risks associated with the dispersion of aerosolized pathogens from such sources has still not been fully determined, but some presumptive evidence indicates that increased enteric disease among nearby residents may be associated with such wastewater irrigation practices [1].

In order to properly design and evaluate wastewater land application facilities involving sprinkler irrigation techniques or other wastewater treatment or disposal works that may be associated with the aerosolizing of pathogenic microorganisms, it is essential to develop and validate a model for predicting downwind microbial concentrations based upon concepts of atmospheric diffusion and microbial die-away as a function of various meteorological and climatological parameters.

Conventional atmospheric diffusion models [2, 3] generally are designed

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to predict the downwind concentration of conservative aerosols as a function of a) wind velocity; b) atmospheric stability; c) height of source above ground level; and d) concentration in the source stream. In the case under study we have attempted to relate to the special problem of aerosolized viable bacterial die-away under varying climatic conditions.

It is our objective to apply and validate a relatively simple model for estimating the potential concentration of live microorganisms at the breathing zone level downwind from a continuous point source. The first essential steps have been the development of a methodology for determining the bacterial die-off constant, λ , under field conditions as a function of key environmental parameters, and to validate the atmospheric diffusion model under the specific conditions of spray irrigation. These two matters are the objective of this study.

An Atmospheric Diffusion Model

Modeling efforts for viable aerosols to date have used atmospheric dispersion models from pollution monitoring programs [2, 3]. These models are generally adequate in their ability to predict aerosol behavior and plume distribution in relation to meteorological conditions. However, they are inadequate in that they fail to take into account the loss in aerosolized microorganism concentration caused by biological death. The Gaussian plume model allows atmospheric dispersion prediction for inert particles $[X(x,y,z)]$ from a point source, when the height of the source (H) and the meteorologic conditions are known [3].

$$X(x,y,z) = \frac{Q}{2\pi\sigma_y\sigma_z u} \exp\left[-\frac{1}{2}\left(\frac{y}{\sigma_y}\right)^2\right] \left\{ \exp\left[-\frac{1}{2}\left(\frac{z-H}{\sigma_z}\right)^2\right] + \exp\left[-\frac{1}{2}\left(\frac{z+H}{\sigma_z}\right)^2\right] \right\} \quad (1)$$

where X = number of particles per m^3 of air
 Q = number of particles emitted from the source per second
 u = mean wind velocity, m/sec
 σ_y, σ_z = diffusion factors in y, z planes and are functions of meteorological conditions, and the downwind distance from the source.

The height of the source H and coordinates x, y, z are all in meters. In this model the following assumptions are made: (1) The plume has a Gaussian distribution in the vertical and horizontal planes; (2) all the particles are completely reflected from the earth; (3) the source emits at a fixed rate; (4) wind velocity is constant for a given time and place; (5) the surface is flat; (6) the aerosol particles are smaller than 20 microns; (7) the relative velocity between the wind and the source is negligible as is the diffusion downwind.

Biological Death Modification of the Diffusion Model

Several modeling approaches have been proposed to describe the creation of viable aerosols from wastewater sources and to predict aerosol downwind concentrations [4, 5, 6, 7, 8]. The maximum number of live particles remaining in the atmosphere after a certain period of time (t) depends on the physiological sensitivity of the cells and on atmospheric conditions. By knowing the biological death (BD) constant (λ) in a number of specific cases, one of the approaches [6] proposes to modify equation (1) to account for these factors:

$$X(x,y,z)_{BD} = X(x,y,z) \exp(-\lambda t) \quad (2)$$

where:

$X(x,y,z)_{BD}$ = concentration modified according to the microbial die-off rate
 t = average aerosol age, in seconds
 λ = microbial die-off constant in sec^{-1} , determined experimentally for different microorganisms under various spray irrigation and atmospheric conditions

It is possible to approximate t by means of $x/\bar{u}$ when equation (2) becomes:

$$X(x,y,z)_{BD} = X(x,y,z) \exp\left(\frac{-\lambda x}{\bar{u}}\right) \quad (3)$$

An additional modification is required when only a part of the emission becomes an aerosol, as occurs with a sprinkler operating under windy conditions. Thus equation (3) will change to:

$$X(x,y,z)_{BD} = X(x,y,z) \cdot E \cdot \exp\left(\frac{-\lambda x}{\bar{u}}\right) \quad (4)$$

where E = the aerosolization efficiency factor; i.e., the fraction of the sprayed liquid which actually enters the atmosphere as an aerosol. The microbial die-off rate (λ) in the natural atmosphere is a dynamic function of many biological and environmental variables and presents many complex experimental problems measuring it in situ - under field conditions. Laboratory measurements of the microbial die-off rate in the air as a time function of a number of variables have been done mostly in steady state conditions and only rarely under natural dynamic conditions [9].

METHODS AND MATERIALS

Experimental Site

The site is a flat area situated 750 meters above sea level at a distance of 10 kilometers from Jerusalem. The site is a characteristic low shrubland composed mostly of *Poterium spinosum* bushes up to 50 cm tall.

Sprayer

The Minute Man Jet Fog Sprayer (International Industries, Inc., Chicago, Ill.) was used. This sprayer creates drops in a size range of 18-50 microns. The height of the sprayer was 1 meter from the ground and the plume was about 1.5 meters above ground level.

Cultivation and Labeling of Bacteria

E. coli K12 bacteria were grown at 37°C in M-9 medium for one night. In the morning the bacteria were diluted in the same medium to an optical density of 4 Klett units and then shaken for two hours at 37°C. To this bacterial suspension was added L-leucine-4.5-T with a specific activity of tritium of 30.3 Ci/mmol (Nuclear Research Center, Negev, Beer Sheva, Israel), to achieve a final concentration of $2 \cdot 10^{-4}$ mCi/ml and was then shaken for another hour. The bacteria were then washed three times in cold medium and centrifuged at 10,000 rpm for 15 minutes each time. The labelled bacteria were taken, packed in ice, to the

field at a final volume of 20 ml. The average efficiency of the labeling was 4.7×10^3 bac./cpm $\pm 3 \times 10^3$ bac./cpm; the counting efficiency was 29.6% $\pm 4.0\%$ and the background was 39 cpm ± 13 cpm.

Experimental Procedure

The labelled bacteria suspension was diluted in 2 liters of distilled water containing 1000 ppm of beef extracts or 2 liters of sterilized sewage. Spraying time varied between 20 and 60 minutes. The high volume air sampler stood at a distance of 20 or 40 meters downwind from the sprayer, according to the wind conditions and topography of the site. Sampling continued until the completion of spraying. During each experiment a sample was taken of the spray immediately adjacent to the sprayer nozzle. At the end of each experiment a sample of the bacterial suspension was taken from the sprayer container.

Airborne bacteria were sampled by a high volume Scrubber-Cyclone type sampler [10]. The collecting fluid was distilled water with 1% beef extract at a flow capacity of 3 ml per minute. The air sampling rate was 500 liters per minute. The samples were preserved on ice until they were brought to the laboratory where bacteriological and radioactive tests were performed on the same day. Emission rates were in the range of 26 to 11,357 cpm/sec. Samples were taken 1m from the ground.

Microbiological and Radioactive Analysis

In all the samples taken, the concentration of live bacteria was determined on endo agar (Difco) bacteriological plates, as described previously [11]. In order to determine the amount of radioactivity in the spray fluid and in the spray adjacent to the sprayer nozzle, 10 ml of these fluids were first filtered through a 25 mm filter with porosity of 0.45 microns (Millipore filter HAWPO 2500) and then washed with distilled water and counted by a liquid scintillator counter (Packard) with scintillation liquid based on Toluene-Triton. The radioactivity in the collecting fluid was determined in the same way, but using the total volume.

Meteorological Data

The wind velocity was measured by a hand anemometer at the height of two meters above ground level. The temperature, and relative humidity were also measured in the field. Solar radiation was measured by a meteorological station in Jerusalem. The atmospheric stability classes were determined according to Pasquill's table (3). All experiments were made during the period July-September; these months have very little variability in meteorological conditions and no rainfall.

The Calculation of λ ; the Microbial Die-Off Constant

Definitions:

In the collecting fluid (sampler):

- L = concentration of live bacteria (bac./ml)
- C = radioactivity concentration (cpm/ml), total live and dead bacterial count.

In the spray adjacent to the sprayer nozzle:

- L' = concentration of live bacteria (bac./ml)
- C' = radioactivity concentration (cpm/ml), total live and dead bacteria count.

R = the relation between the ratio of the concentrations of live bacteria in the collecting fluid and in the spray adjacent to the sprayer nozzle, to the ratio of the radioactivity in the collecting fluid and in the spray adjacent to the sprayer nozzle.

$$R = \frac{L/L'}{C/C'} = \frac{L}{L'} \times \frac{C'}{C} \quad (5)$$

Thus the percentage of surviving bacteria is $R \times 100$ and the percent of dead bacteria is $(1-R)100$.

On the assumption that biological decay is the same as radioactive decay, $[N = N_0 \exp(-\lambda t)]$, we may write λ as:

$$\lambda = \frac{\ln(N_0/N)}{t} \quad (6)$$

where we denote:

N_0 = ratio between the live bacteria concentration and the concentration of the radioactivity in the spray adjacent to the sprayer nozzle

N = the ratio between the live bacteria concentration and the radioactivity concentration in the collecting fluid.

By using equation (5), we can rewrite equation (6) as:

$$\lambda = \frac{\ln(N_0/N)}{t} = \frac{\ln(L'/C')(C/L)}{t} = \frac{\ln(1/R)}{t} \quad (7)$$

It is also possible to calculate λ from the slope of a graph plotting R as a function of t , assuming a straight line relationship exists for the major portion of the slope after the rapid reduction in bacterial concentration that occurs in the first 10-20 sec.

RESULTS

Sixteen experiments, using distilled water with 1000 ppm of beef extract as spray fluid, were conducted on eight different days. Eight of these experiments were conducted in the early morning hours at or before sunrise, with essentially no radiation. An additional 15 experiments were conducted with sterilized sewage as the spray fluid, on eight different days. Seven of these experiments were conducted in the early morning hours.

Figure 1 shows the correlation between the predicted concentrations of the radioactivity as it was calculated from equation (1) for the emission and sampling at ground level* along the central axis of the plume, to the measured concentrations of the radioactivity in the air for the series of experiments using sterilized sewage ($r = .68$). The coefficient of correlation between the predicted and observed radioactivity in the air is $r = .30$ for distilled water with beef extract. However, considering the limitations of the field experiments, Pasquill's equation provides a reasonably good description of the dispersion of the bacterial plume created by the spray, neglecting die-away.

When comparing the percent of live bacteria to the total bacteria in the sprayer reservoir and in the air adjacent to the nozzle, an average loss of 34% occurred, indicating that the act of spraying and aerosolization did not significantly affect the bacterial viability. For this calculation R was defined as the relation between the ratio of con-

*1 m from surface.

centrations of live bacteria in the spray adjacent to the sprayer nozzle and in the sprayer reservoir, to the ratio of the radioactivity in the spray adjacent to the sprayer nozzle and in the sprayer container. No significant differences in the sampler efficiency were noted for diluted or concentrated radioactive aerosols ranging between 10^2 and 10^6 bacteria per m^3 of air. The lowest aerosol concentration which was sampled was 0.2 cpm per m^3 which is equivalent to 940 bacteria per m^3 . The highest concentration was 318 cpm per m^3 which is equivalent to 1.5×10^6 bacteria per m^3 of air.

Early morning experiments were different in a number of factors from experiments conducted in the afternoon in addition to radiation which was essentially zero at sunrise and 70 ± 10 g/cal/cm² in the afternoon. The mean wind velocity in the morning was 0.9 meters per second in contrast to 2.0 meters per second in the afternoon. The mean temperature in the morning was $17^\circ\text{C} \pm 0.6^\circ\text{C}$ as compared to the mean afternoon temperature of $26.5^\circ\text{C} \pm 2.0^\circ\text{C}$. The mean relative humidity in the morning was $98\% \pm 1\%$ in contrast to $50\% \pm 10\%$ in the afternoon. The morning hours were characterized by greater atmospheric stability, class B-C, in contrast to class A-B observed during the afternoon.

As seen in Figures 2 and 3, in the afternoon experiments there is a very rapid die-away rate resulting in a 3-4 log cycle reduction in bacterial count in 30-40 seconds with a 90% reduction (T_{90}) about every 10 seconds. In the morning experiments a much slower rate of bacterial die-away was detected with a 1-2 log cycle reduction in bacterial count in about 130 seconds, with a T_{90} of about 100 seconds.

The bacterial decay rate λ as calculated from the slope of the regression lines in Figures 2 and 3 are 9.4×10^{-3} seconds⁻¹ for the morning and 6.4×10^{-2} seconds⁻¹ for the afternoon with distilled water and beef extract while for the series with sterilized sewage $\lambda = 8.1 \times 10^{-3}$ for the morning and 6.6×10^{-2} for the afternoon. The mean λ for both experimental series is 8.8×10^{-3} seconds⁻¹ for the morning and 6.6×10^{-2} seconds⁻¹ for the afternoon.

The mean λ as calculated according to equation (7) excluding the points during the rapid die-away stage in the first 10-20 seconds was 2.6×10^{-2} for the morning and 3.4×10^{-1} for the afternoon with distilled water, while for the series with sterilized sewage it was 2.6×10^{-2} for the morning and 2.3×10^{-1} for the afternoon. Due to limitations in the experimental design, it is felt that the λ calculated by the graphical method is the more representative figure.

In general the experiments with distilled water and beef extract as the spray fluid showed less variance than those with sterilized sewage. The correlation coefficient (r) for log R and t for the distilled water series was $r = -.90$ for the morning and $r = -.68$ for the afternoon while with sterilized sewage r was $-.56$ and $-.60$, respectively. This is based on the assumption that a straight line relationship exists for the portion of the curve after the first 10 or 20 seconds.

DISCUSSION

A relatively simple model is proposed based on Pasquill's atmospheric dispersion equation [3] modified for biological death and aerosol efficiency of the source. In order to determine a microbial die-off constant (λ), a sprayer was selected whose aerosolization efficiency was presumed to be close to 100% and the range of the particle size to be

between 18 and 50 microns.

The observed decay of a biological aerosol results from two major causes: (1) biological decay (biological inactivation, die-off and loss of ability to form colonies or plaques under given experimental conditions); (2) physical decay (gravitational settling, impact on surfaces and clumping into larger particles). Tracers used to define physical decay include fluorescent dyes such as Rhodamine B [12], sodium fluorescent (uramine) [13], and resistant bacteria such as *Bacillus subtilis* var. *niger* spores. The use of bacteria labelled by radioactive isotopes used in this study enables the determination of biological and physical decay at the same time. The disadvantage in using fluorescent dyes as tracers is that some of the particles created by the dyes are smaller than the aerosolized bacteria. Consequently they do not possess the same physical decay rates as aerosolized bacteria. An additional disadvantage is that fluorescent dyes are likely to be toxic to microorganisms, particularly in aerosols exposed to the sun [14]. Bacterial spores, despite their great resistance, are likely to germinate following the impingement processes and may subsequently die as a result of difficult conditions during sampling time.

In this research, bacteria labelled with tritium were used, overcoming many of the problems faced by others, since they themselves constitute simultaneously both physical and biological tracers. *E. coli* bacteria were selected because they are common enteric bacteria found in wastewater and are relatively easy to handle.

Calculations of physical dispersion in the air verified the assumption that the bacterial plume created by the sprayer is generally describable by the Pasquill equation (Figure 1). The immediate spray burst caused by the sprayer itself apparently had limited influence on the bacterial die-off. The sampler efficiency remained constant at aerosol concentrations ranging between 10^2 and 10^6 bacteria per m^3 of air. Calculations of the biological decay rates were made by determining R , the relationship between the ratio of radioactivity detected downwind in the high volume air sampler and at the sprayer nozzle to the ratio of live bacteria in the air sampler and at the sprayer nozzle.

There were a number of technical limitations, such as the availability of only one high volume air sampling device, which required sampling at the sprayer nozzle at time t_0 in a different manner from the sampling done at a distance downwind from the sprayer at time t . There also were topographical limitations which complicated determination of the main wind direction and dictated relatively short sampling distances. Despite all of the above-mentioned limitations, atmospheric conditions were shown to exert an important influence on the biological decay rate. In Figures 2 and 3, a considerable difference is apparent between morning time experiments and afternoon time experiments, in regard to R and λ . The morning time experiments were characterized by greater atmospheric stability, lower wind velocity, lack of solar radiation, lower temperatures, and very high relative humidity (98%). The time required for a 90% reduction in bacterial count was about 100 seconds, while in the afternoon experiments, the T_{90} was about 10 seconds.

Since atmospheric stability and wind velocity are primarily expected to influence physical dispersion with little known influence on bacterial die-away rates, it can be assumed that with the methods used, which

allowed us to neglect physical dispersion. The differences in biological decay are attributable mainly to those environmental factors thought to directly affect bacterial viability, i.e. temperature, solar radiation and relative humidity (R.H.)

In our previous field studies using a marker bacteria [15] we have shown that temperature in the range between 20-27°C had no significant effect on bacterial aerosol viability. Global irradiation in the range 10-80 gm. cal./cm² was shown to be weakly negatively correlated with *E. coli* aerosol survival, while R.H. between 50 and 85% was shown to have a strong positive correlation with bacterial survival. Based on these previous studies we found that at a R.H. of about 90% we would expect a 10 times higher concentration of live bacteria in the air at a given sampling point than at a relative humidity of 50%. This leads us to assume that high relative humidity provides environmental conditions very much more favorable to bacterial survival which may prove to be the dominant factor in determining the die-away rates of aerosolized bacteria. Solar radiation undoubtedly plays an independent role, particularly under conditions of low R.H. It must be remembered that in the early morning experiments radiation was essentially zero, so that it can be assumed that under those conditions and under night-time wastewater irrigation in general the R.H. may provide a possible single parameter capable of predicting, within a given temperature range, the λ required for the model under consideration. Others have also shown the importance of R.H. on bacterial aerosol survival [7, 8].

The λ determined by the graphic method for the afternoon experiments of 6.6×10^{-2} seconds⁻¹ was undoubtedly influenced by a combination of detrimental environmental factors, including the high solar radiation and low relative humidity, while the λ determined in early morning experiments of 8.8×10^{-3} seconds⁻¹ may represent primarily the influence of the high relative humidity (98%) since radiation was zero. This figure may represent a first rough approximation of the field biological decay rate of coliforms under night-time sprinkler irrigation conditions in periods of very high relative humidity.

Since the purpose of a model is to provide a tool for predicting downwind concentrations under the least favorable conditions and since sewage sprinkler irrigation as well as conventional sewage treatment processes normally function at night, neglecting the effect of radiation is not an unreasonable assumption. However, this possible simplification is premature at this time and awaits confirmation by further field experiments. Pathogens, including viruses, may have considerably slower die-away rates and the use of the λ for coliforms may be misleading. The very rapid die-away of bacteria during the first 10-20 seconds, particularly under afternoon conditions of both low R.H. and high global radiation, may be associated with the more rapid death of a highly susceptible portion of the bacterial population, or those appearing as single more exposed bacteria rather than as bacteria in more protective clumps or embedded in organic particles. Rapid sedimentation of the larger aerosols formed might also contribute to this phenomenon. It can be assumed that the λ varies for these different stages of the decay curve, thus explaining the difference between the calculated figure and that obtained by the graphical method on the straight line portion of the slope. Recognizing the limitations of this experimental design, using only one high volume sampler in each experiment we have assumed that the λ by the graphic method is the figure that more closely approximates the bacterial die-away

rate over longer distances.

In order to further elucidate the questions dealt with in this study, it is essential that λ be determined under a wider range of environmental conditions, at numerous points simultaneously and for different microorganisms of differing inherent environmental resistance. The goal should be the development and validation of the predictive model for viable aerosolized microorganisms that can be used to aid in the design, siting, and evaluation of wastewater irrigation and wastewater treatment facilities. It is felt that the methodology developed in this study based on the use of tritium-tagged bacterial tracers may help to overcome some of the obstacles that have blocked progress in this area to date.

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Figure 1. Correlation Between
Observed Aerosol Concentration as cpm/m^3
and Predicted Concentration
According to Pasquill's Equation
(Sterilized Sewage)

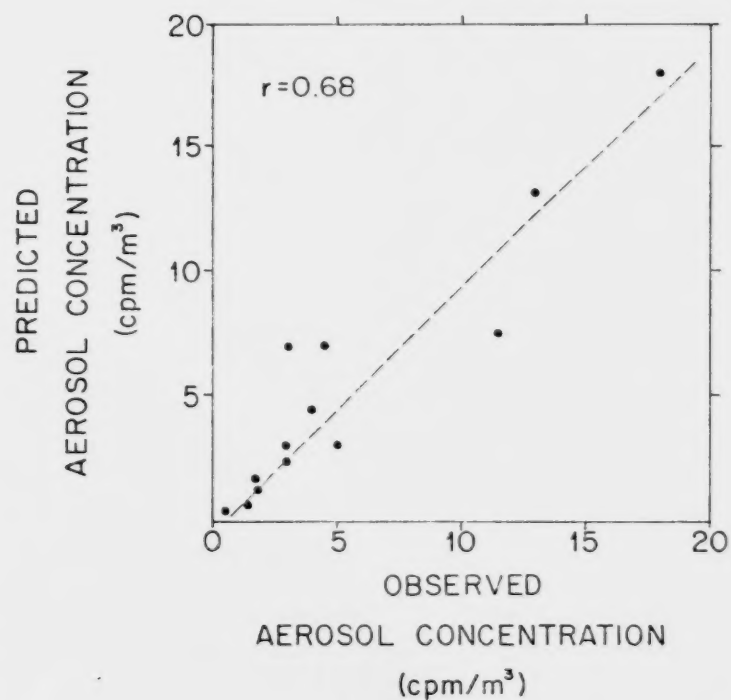


Figure 2. Bacterial Aerosol Reduction
As a Function of Time (t)
—Distilled Water + 1000ppm Beef Extract—

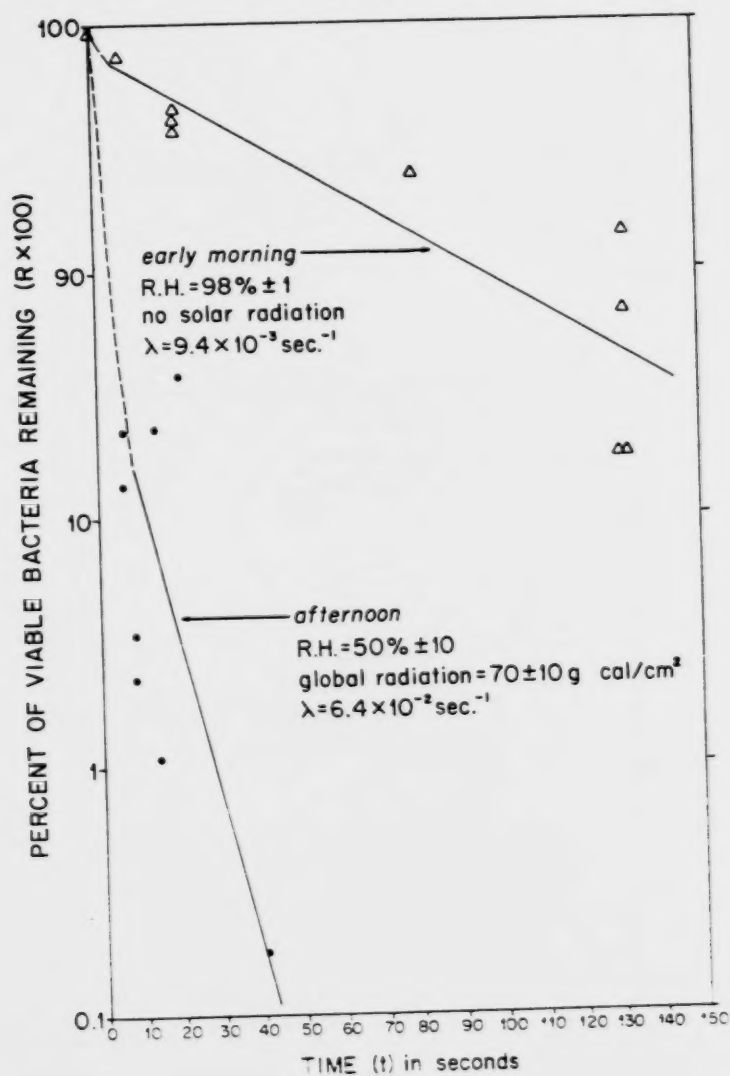
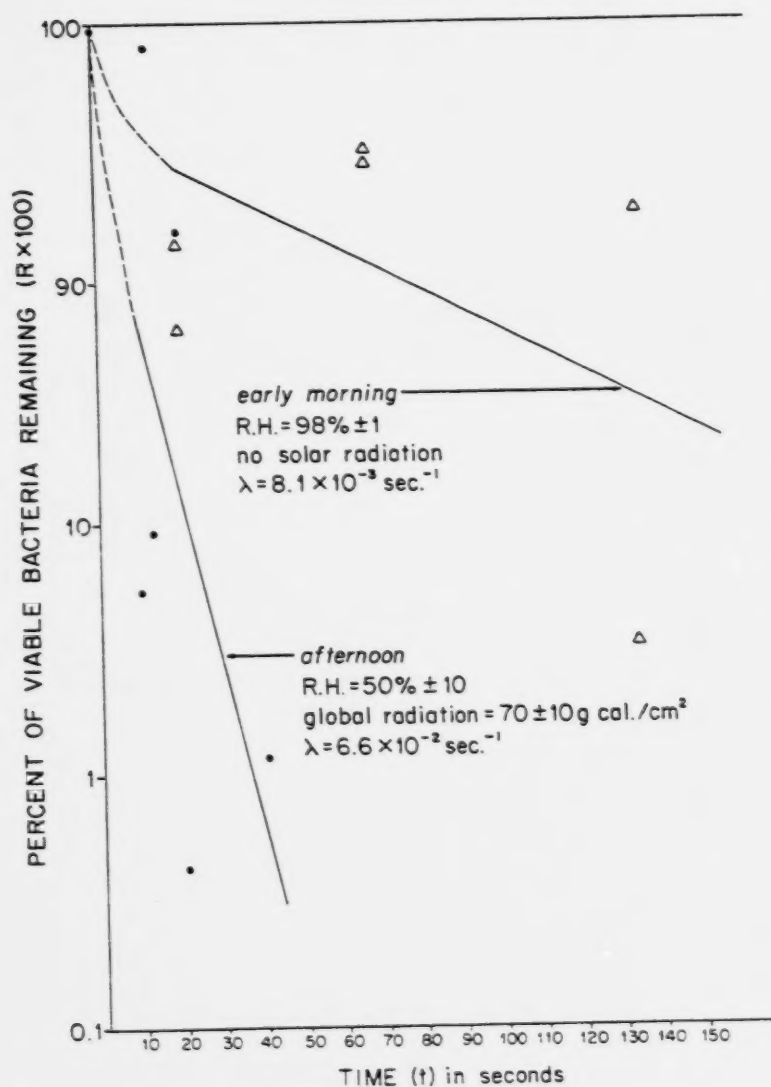


Figure 3. Bacterial Aerosol Reduction
As a Function of Time (t)
—Sterilized Sewage—



OCCURRENCE OF MUTAGENS/CARCINOGENS IN MUNICIPAL WASTEWATERS AND THEIR
REMOVAL DURING ADVANCED WASTEWATER TREATMENT

J. Saxena
Senior Scientist
Center for Chemical Hazard Assessment
Syracuse Research Corporation
Merrill Lane, Syracuse, NY 13210

D. J. Schwartz
Research Associate
Center for Chemical Hazard Assessment
Syracuse Research Corporation
Merrill Lane, Syracuse, NY 13210

M. W. Neal
Research Associate
Center for Chemical Hazard Assessment
Syracuse Research Corporation
Merrill Lane, Syracuse, NY 13210

ABSTRACT

The physical-chemical treatment processes used at the Piscataway, MD pilot wastewater treatment plant were examined for their ability to remove or introduce mutagenic/carcinogenic substances from wastewater using Ames *S. typhimurium* assay. Liming and/or recarbonation steps appeared to introduce substances which were mutagenic to strain TA 1535 and TA 100. The mutagens introduced and those originally present in the influent were not efficiently removed during treatment since the mutagenic activity of final AWT effluent was no less than that of the influent. Polyurethane foam, XAD and solvent extraction succeeded in recovering only a part of the mutagenic activity detected in wastewaters. The recovered activity was located predominantly in basic, ether insoluble, strong acid and water soluble fraction. Among the various concentration methods, polyurethane foam appeared to be more selective in recovering mutagens/carcinogens. The neutral fraction was generally toxic to *Salmonella* and could not be assayed for mutagenicity without subfractionation into aliphatic, aromatic and oxygenated classes. Nearly all the mutagenic activity of this fraction was located in the oxygenated fraction. The chemical components of wastewaters which could be recovered by adsorption on solid sorbent such as polyurethane foam, XAD resin or extracted by solvent appeared to be efficiently removed during physio-chemical treatment. However, some mutagens detected in the unconcentrated wastewaters could not be recovered by any of the concentration methods employed. These mutagens, probably polar in nature, were also the ones not adequately removed by AWT treatment.

INTRODUCTION

As clean water resources become scarce and the need to protect our environment becomes urgent, wastewater reclamation will be increasingly practiced. Advanced Wastewater Treatment (AWT) processes are used following conventional treatment, not only to protect certain lakes and rivers but also to produce high quality water for reuse. Reuse

applications of wastewater include well established uses such as irrigation, formation of recreational lakes, industrial uses and also others which are only beginning to be considered such as ground water recharge, and direct domestic reuse. Unintentional direct reuse of wastewater occurs widely at the present time via consumption of drinking waters drawn from polluted rivers and lakes.

Prior to considering reuse of renovated wastewaters for potable purposes, the potential hazards from the consumption of renovated wastewaters must be assessed. Wastewaters contain many hazardous chemicals for which currently no criteria exist. The advanced wastewater treatment technology available today leaves many potentially hazardous chemicals unchanged and in some instances introduces new and potentially toxic chemicals (1,2). Further complications arise from the fact that residual or introduced trace pollutants vary from one situation to the next depending upon the nature of the industrial waste which enters the sewage system.

In view of the growing recognition that the majority of human cancers are due to the presence of chemicals in the environment, the possible occurrence of carcinogenic chemicals in renovated wastewater is of particular concern. In the present investigation the physical chemical treatment process used at the Piscataway, MD pilot wastewater treatment plant was examined for its ability to remove (or introduce) mutagenic/carcinogenic substances from wastewaters. Further, the distribution of the detected activity among various classes of chemical compounds was studied in an attempt to identify the active fraction(s).

EXPERIMENTAL

Piscataway, MD Model AWT Plant (High Lime Process)

This is a 5 million gallon per day plant. The treatment consists of lime addition, recarbonation, dual media filtration, breakpoint chlorination and activated carbon filtration. A flow diagram of the treatment steps used at this plant is shown in Fig. 1.

Samples and Sampling Procedure

Three sampling points were selected for these studies. These included influent for AWT, which is the secondary effluent from conventional treatment process, effluent after liming and recarbonation step (henceforth referred to as intermediate effluent), and tertiary effluent.

Mutagenicity studies on the unconcentrated wastewaters were performed on twenty-four hour composite samples collected in glass container with the help of an ISCO automatic composite sampler. The rate of sample collection was maintained at approximately 250 ml every 30 min. All samples were kept in wet ice during the compositing period and transportation. In view of the generally recognized quality fluctuations of the influent wastewaters, two samples which were collected several weeks apart were tested. Samples were sterilized by serial filtration with 1200 nm, 450 nm and finally 200 nm membrane filters (Millipore Corp.).

The concentration of trace organics from wastewaters by XAD and polyurethane foam was performed at the treatment site using a portable apparatus which allowed maintenance of optimum conditions for recovery. Following exposure to an adequate volume of wastewater, the sorbent

material was brought to the laboratory for further processing. This helped overcome the difficulties associated with shipping large volumes of samples for concentration, and possible deterioration of samples during transportation and storage. For recovery of organics by liquid-liquid extraction, wastewater samples were brought to the laboratory. These samples were preserved in transit by saturating with 2% methylene chloride and kept cold.

Recovery of Trace Organics from Wastewater

In order to recover a wide variety of organic contaminants from wastewaters, three concentration methods were employed: Polyurethane foam plugs, macroreticular resin, and liquid-liquid extraction employing 15% methylene chloride in hexane. The wastewaters were filtered through glass wool prior to subjecting them to any concentration procedure. Liquid extractions were performed by mixing wastewater overnight with methylene chloride-hexane (70 ml solvent/l wastewater).

The procedure for recovery of organics by polyurethane foam plugs was utilized as reported in the literature, without any modification (3,4). This involved passing water through flexible polyurethane foam plugs at $62 \pm 2^\circ\text{C}$ and maintaining a flow rate of 250 ± 10 ml/min. Four foam plugs were used for recovering organics from 10l of wastewater. The sorbed compounds were eluted with benzene. The adsorption capabilities of macroreticular resin were examined using the secondary effluent from the local treatment plant and the best operating conditions defined for the recovery of total organic carbon. The best results were obtained when two columns (1 x 10 cm) were used in series - the first containing XAD-4 and the second XAD-8. Wastewater temperature and flow rates also affected the efficiency of retention. Optimum temperature was 50°C and optimum flow rate was 100 ml/min. Two 1 x 10 cm XAD-4 and XAD-8 columns connected in series permitted collection of organics from 5l of water. The total organic carbon retention under these conditions ranged between 18-20% for the local secondary effluent (total organic carbon 35 ppm). The sorbed compounds on the resin were recovered by Soxhlet extraction for 18 hours with methanol. The recoveries obtained were better with methanol than with ether, acetone or methylene chloride.

Fractionation of Recovered Organics into Major Chemical Classes

The crude organic concentrates were separated into major classes of organic compounds according to their solubility under acidic, basic and neutral conditions using a modified Shrier-Fusion separation technique (5). The major classes of compounds isolated were: ether insolubles, water soluble, bases, amphoterics, strong acids, weak acids, and neutrals. The neutral fraction was subfractionated into aliphatic aromatic and oxygenated fractions by chromatography or silica gel. The complete separation scheme is shown in Fig. 2.

Mutagenicity Assay

Ames *Salmonella typhimurium* assay employing strain TA 100 and TA 1535 as base pair substitution mutants, and strain TA 98 and TA 1538 as frameshift mutants was used in these studies. To facilitate detection of low concentrations of mutagens suspected in unconcentrated wastewaters, the bioassays were performed by preparing the assay media (base agar layer) with the filter sterilized wastewaters under investigation. Thus, suspected mutagens would be present in the bottom agar, whereas bacteria and mammalian activation systems (where added) would be in the

agar overlay. This modification of the assay permitted incorporation of up to 20 ml wastewater/assay (70% v/v). Metabolic activation of mutagens was achieved by incorporating rat liver homogenate plus necessary cofactors in the assay. The procedure used for preparation of the liver homogenate and liver homogenate-cofactor mix was the same as described by Ames (6). Mutagenicity assays on various unconcentrated wastewaters collected at a particular date were carried out at the same time together with distilled demineralized water controls for spontaneous reversion rate and positive mutagenesis controls. The results for unconcentrated wastewaters are expressed as averages of triplicate determinations.

For mutagenicity assays on crude wastewater concentrates and separated fractions, samples were evaporated to dryness and reconstituted with dimethylsulfoxide. Preliminary experiments showed that liquid suspension assay, which involved exposing the organism to the test mixture in a liquid suspension prior to plating in agar medium, detected mutagens more effectively than the conventional plate incorporation assay or spot test. Therefore, liquid suspension assays were routinely used in these studies. The incubation medium contained 1 ml minimal medium, 0.5 ml S-9 mix where added (6), 0.1 ml of appropriate tester strain (5×10^8 cells), 4 μ g histidine and 6 μ g biotin, and appropriate amounts of the test organic mixture. Identical control tubes were set up without the test organic mixture. Following a 60 minute incubation at 37°C, 3 ml soft agar containing 8 μ g histidine and 12 μ g biotin/ml was added and the contents were overlaid on minimal glucose agar. Mutagenicity was shown by an increase in the number of revertants over spontaneous (control). The assays were carried out in duplicate.

Assay of Histidine in Wastewaters

Wastewaters were assayed for the presence of histidine using an enzymatic assay which involved conversion of histidine to urocanic acid by *l*-histidine ammonia lyase (E.C. 4.3.1.3) and its measurement by increase in absorbance at 277 nm (7).

Total Organic Carbon Determination

TOC in wastewaters and organic concentrates was determined by Oceanography International Corporation's TOC analyzer using their glass ampoule technique.

RESULTS AND DISCUSSION

Identification of every compound of concern in complex uncharacterized medium such as municipal wastewater is an extremely difficult task. Even if identification could be accomplished, nothing will be revealed from the data with respect to any health effects linked to their presence in wastewaters except in a few cases where information is available in the literature. An alternate strategy is to use rapid *in vitro* bioassays for detecting presumptive biological activity in wastewater samples and locating active fractions, and then subjecting the active fraction to physio-chemical analysis to fix molecular composition. This strategy would eliminate the need for laborious analysis of samples that do not, in fact, contain hazardous compounds. In addition, this will enable detection of unknown biologically active compounds. In the present investigation we have utilized this approach to identify mutagenic/carcinogenic components of municipal wastewater and to examine their fate during AWT treatment.

The analysis of mutagenicity/carcinogenicity of municipal wastewaters is complicated by many technical problems. Wastewaters, like any complex environmental sample, contain mutagenic/carcinogenic substances in a matrix of innocuous and toxic substances. These substances may inhibit or modify the mutagenic response. Consequently, a clear mutagenic response and a linear dose response curve as one generally finds with pure compound may not always be seen. The low levels of mutagenic/carcinogenic compounds suspected to be present in wastewaters often necessitate a preconcentration step prior to their detection. A further problem is caused due to the presence of histidine in wastewaters. Municipal wastewaters, due to their biological origin, contain many amino acids including small quantities of histidine (8). Histidine interferes with the Ames *Salmonella* assay by increasing the number of spontaneous revertants. This artifact may be confused with a mutagenic response.

Mutagenic Activity of Unconcentrated Wastewaters

Secondary as well as AWT effluent showed significant mutagenesis in the base pair substitution mutants TA 100 and TA 1538 of *S. typhimurium* (Fig. 3,4). The mutagens responsible for the effect were inactivated to a large extent by the presence of mammalian liver enzymes in the assay. The wastewaters failed to revert the frameshift mutant TA 98 but exhibited strong toxicity towards this strain, decreasing the spontaneous reversion rate as much as three-fold in some samples. The inability to detect frameshift mutagens may be linked to the high sensitivity of this strain towards toxicants in wastewaters. Decreasing the volume of wastewaters in the assay relieved the inhibition without inducing mutagenic response.

In view of the low levels of mutagenic activity observed, it became important to determine if histidine was present in these wastewaters and if so whether histidine was artifactually responsible for the increased number of revertant. Assay of the wastewaters for histidine by a specific enzymatic method having a detection limit of 1 µg/ml revealed that histidine was not present in measurable amounts. Furthermore, wastewaters spiked with <1 µg/ml histidine failed to exhibit any significant influence on the spontaneous reversion rate. Thus wastewater-induced increase in the reversion rate was true mutagenesis and not due to histidine.

Chemical mutagens that enter the human body through the diet or in other ways are often excreted unchanged as conjugates. Thus the possibility exists for the presence of some mutagens in municipal wastewaters as conjugates such as glucuronides. Since glucuronides are not split by the enzymes in *Salmonella* or in the rat liver homogenate, β-glucuronidase should be incorporated in the assay to cause hydrolysis (9,10). Presence of β-glucuronidase in the assay did not result in increased mutagenicity with any of the wastewater samples studied. This demonstrates that any mutagens present were not conjugated as glucuronides.

Comparison of the mutagenic response of AWT influent and AWT mid-point effluent showed that lime addition and/or recarbonation were capable of promoting synthesis of mutagens. Some of these compounds were direct acting, whereas others required metabolic activation. The mutagens formed in these treatment processes were partially removed during breakpoint chlorination and/or carbon filtration. However, the overall concentration of mutagens present in the final effluent was no less than that present in the influent wastewaters.

Recovery of Organics from Wastewaters and Mutagenicity Assay of the Concentrates

The total organic carbon content of the wastewaters and the proportion recovered by various concentration methods are shown in Table 1. The measured levels of organic carbon in the wastewaters were surprisingly low. The samples for TOC analysis were taken after filtration through glass wool and hence the lower TOC levels may be due to removal of particulate organic carbon. The secondary effluent contained 12.2 ppm organic carbon. The intermediate point sample represented 77% reduction in carbon and the carbon level in AWT effluent was reduced by 88%. The percent of carbon recovered by the collection methods varied with the level of carbon in the effluent. Whereas in the case of foam and XAD, the retention efficiency increased with decreasing carbon in wastewater, the reverse was true with solvent extraction. On the basis of carbon recovery, solvent extraction gave the best results with secondary effluent, while XAD was best with intermediate and tertiary effluents.

Mutagenic responses of the recovered organics in *Salmonella* strain TA 100 and TA 1538 are shown in Fig. 5 and 6, respectively. Although polyurethane foam collected the least amount of organic carbon, the recovered carbon possessed mutagenic activity roughly equivalent to that recovered with XAD and solvent suggesting some specificity of foam plugs for recovering mutagens. The organic mixture recovered from AWT influent was mutagenic for *Salmonella* strain TA 100 but not to a significant extent in TA 1538. Following liming and/or recarbonation of the influent the mutagenic response shown by TA 100 remained unaffected but there was a significant increase in mutagenicity with TA 1538. The findings show that liming and/or recarbonation processes have the potential of introducing compounds capable of causing frame-shift mutation. The concentration methods employed failed to recover base pair substitution mutagens shown to be present in the unconcentrated intermediate effluent (Fig. 3 and 4).

The final AWT effluent did not exhibit significant mutagenic activity in any of the *Salmonella* strains tested. This does not show that the mutagens present in influent wastewaters and those introduced during liming and/or recarbonation were effectively removed during the final stage of treatment, since mutagens were detected in unconcentrated samples of AWT effluents. The inability of the concentration methods to recover these mutagenic substances from AWT effluent appears to be responsible for their absence. As the mutagens in unconcentrated wastewaters were detected by preparing the bottom agar with the test wastewater, diffusion of mutagens from the bottom layer to the top layer was necessary. Since the indicator organisms and S-9 mix were added to the top agar layer, only in this way could mutagens be acted upon by liver enzymes, or cause reversion in *S. typhimurium*. Our studies have shown that non-polar compounds such as benzo(a)pyrene which fail to diffuse in aqueous medium were not detected satisfactorily in this system. This may indicate that the mutagens detected in unconcentrated wastewater were polar and therefore probably not efficiently recovered by adsorption on XAD or foam, or by solvent extraction. On the other hand, those mutagens present in influent wastewaters which could be recovered by the above concentration methods were efficiently removed during treatment. The inability of the concentration methods employed to recover mutagens of polar nature combined with the efficient removal of the non-polar mutagens by the treatment processes appear to be responsible for the absence of mutagenic activity in the organic mixture recovered from AWT effluent.

Mutagenicity Assay of the Major Fractions of Wastewater Concentrates

The solubility separation of the crude mixtures into chemical classes and measurement of mutagenic activity of these classes was undertaken in order to identify the active fraction(s). The results are shown in Fig. 7-9. It was noted that the combined activity of all the fractions was generally greater than that of the crude concentrate. The increase may be linked to the enrichment of mutagenic substances in selected fractions separated from toxicants or due to the dilution of toxic material present in the crude mixture. In terms of the quantities and classes of mutagenic substances recovered by the concentration procedures employed, the performance of polyurethane foam plugs was best, solvent extraction being the next, and XAD being the poorest. XAD, however, appeared extremely efficient in recovering toxic components of wastewater. The mutagenic activity recovered by polyurethane foam from AWT influent was distributed largely in basic, strong acid and neutral fraction. The response was noted in strain TA 100 and TA 98 without the presence of the mammalian activation system. Breakpoint chlorination/recarbonation resulted in an increase in the number of fractions which were mutagenic. For example, ether insoluble, amphoteric, and weak acid fractions of polyurethane foam concentrate of AWT influent were nonmutagenic, whereas similar fractions representing intermediate stage effluent were weakly to moderately mutagenic. This provides further support to the earlier observation that liming and/or recarbonation processes introduce mutagens. Similar increase in the number of mutagenic classes was not noted with the activity recovered by solvent extraction or XAD adsorption. The final AWT effluent generally appeared to contain a high concentration of toxicants. These toxicants were not present in the intermediate level effluent and therefore their synthesis probably occurred during breakpoint chlorination or carbon filtration. Significant levels of mutagenic activity were not detected in any of the fractions of AWT effluent. This was in agreement with the earlier findings with crude concentrates.

Neutral fractions recovered by polyurethane foam, XAD or solvent extraction were generally toxic to the *Salmonella* tester strains and therefore were subfractionated into aliphatic, aromatic and oxygenated fraction prior to bioassay. The assay was performed with the concentration of recovered organics equivalent to 50-200 ml for the aliphatic and aromatic fractions, and 10-50 ml for the relatively more toxic oxygenated fraction. The positive mutagenic response by various subfractions of neutral components of wastewater was demonstrated largely in *Salmonella* strain TA 98 in the absence of rat liver homogenate. (Fig. 10-12). Nearly all the mutagenic activity for the neutral fraction of AWT influent was located in the oxygenated fraction. Efficient recovery of these compounds was achieved by solvent extraction. XAD adsorption partially recovered these compounds but polyurethane foam failed to recover them. Oxygenated mutagenic compounds were also detected in the intermediate effluent, however these appeared to be of a different chemical nature, as they could be recovered by polyurethane foam. Furthermore, XAD adsorption and solvent extraction proved ineffective in recovering these mutagens. The data can be explained by suggesting that liming and/or recarbonation processes caused chemical modification of the oxygenated neutrals present in AWT influent. The altered compounds retained mutagenic activity but became resistant to recovery by XAD adsorption or solvent extraction. Alternatively, chemical changes in the wastewaters (e.g. ionic strength, TOC, etc.) resulting from liming/recarbonation may have affected the adsorption properties of the neutral oxygenated fraction.

The final AWT effluent, while it was free of any oxygenated mutagens, contained activity in the aromatic fraction. Since no activity in this fraction was detected in the intermediate effluents, these mutagens appeared to have been synthesized during treatment. The aliphatic and the oxygenated neutrals of the AWT effluent at the concentration used were toxic to the bacteria and therefore it can not be said with certainty if mutagenic compounds were present in these fractions.

The results showed that the physio-chemical AWT method studied was able to adequately remove mutagenic compounds which were susceptible to recovery by adsorption on solid sorbent such as XAD and polyurethane foam, and by liquid-liquid extraction. Mutagens of a highly polar nature which were difficult to concentrate from water were also the ones which were difficult to treat. Indeed, some treatment steps showed a potential for introducing mutagenic substances. Further studies are required to determine the nature of the introduced compounds and to examine their presence in final effluents. The reuse of renovated wastewaters for potable purposes at the present time may present a hazard to health.

ACKNOWLEDGEMENT

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TABLE 1

Recovery of Organic Compounds from Wastewaters
by Different Collection Methods

Sample	Total Organic Carbon (ppm)	Per cent Carbon Recovered by		
		Foam	XAD	Extraction by Methylene chloride- hexane
Secondary Effluent	12.2	2	9.8	24.6
Intermediate Effluent	2.8	8.9	13.6	7.8
Tertiary Effluent	1.4	6.8	46.0	10.0

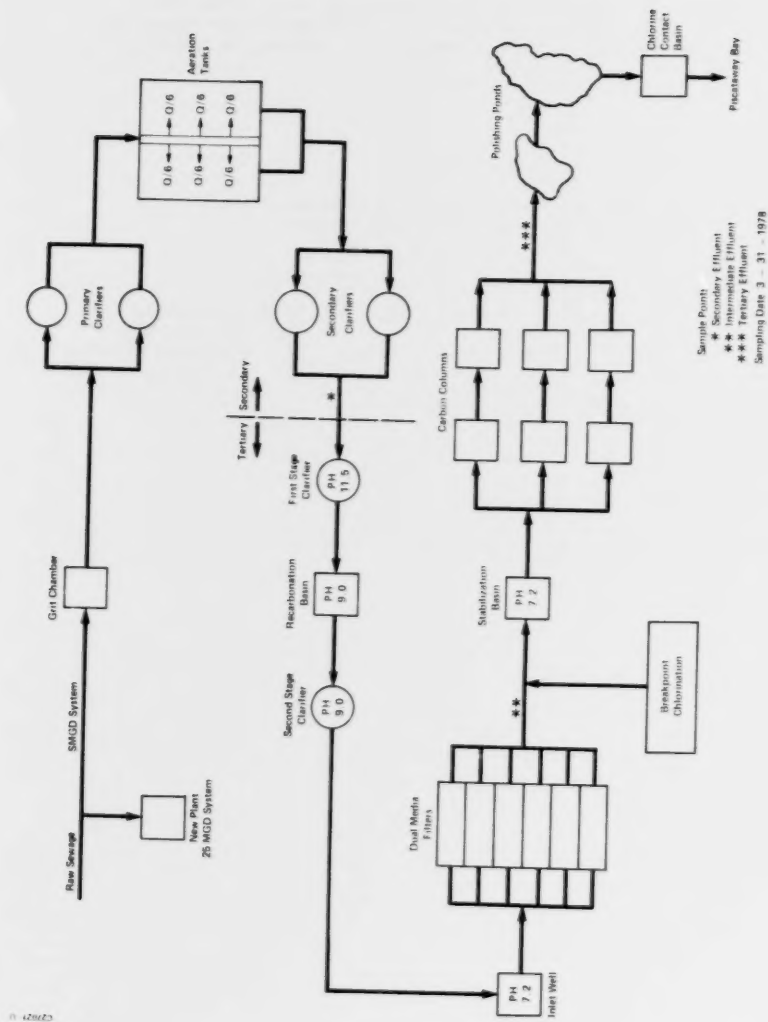


Figure 1. Advanced Wastewater Treatment Plant at Piscataway, Maryland

Flow Scheme for Solubility Separation

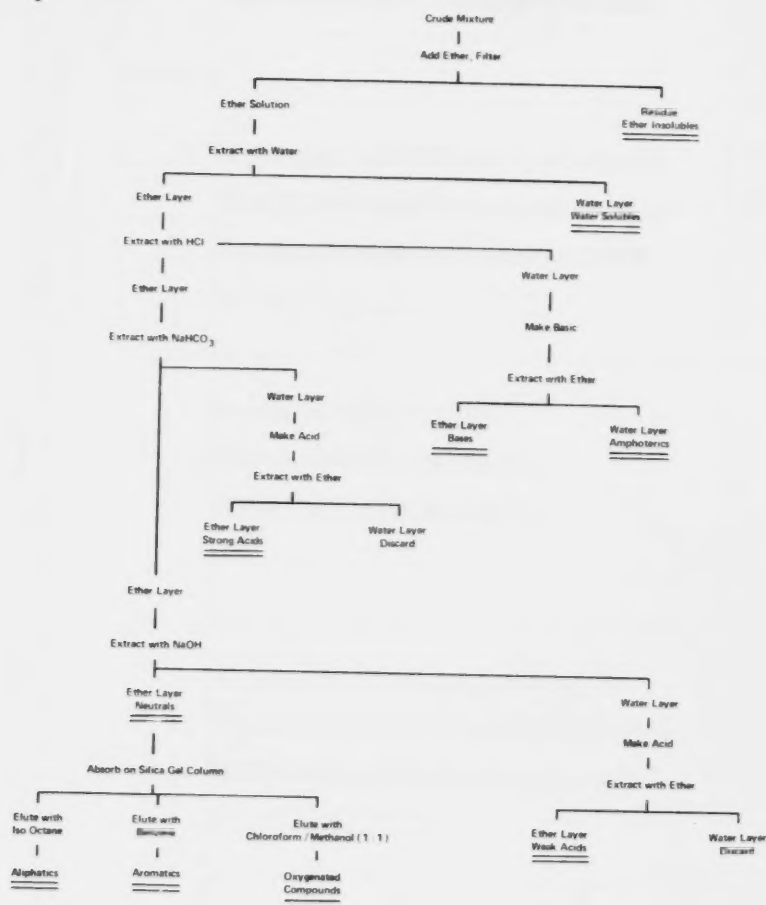


Figure 2. Flow Scheme for Solubility Separation

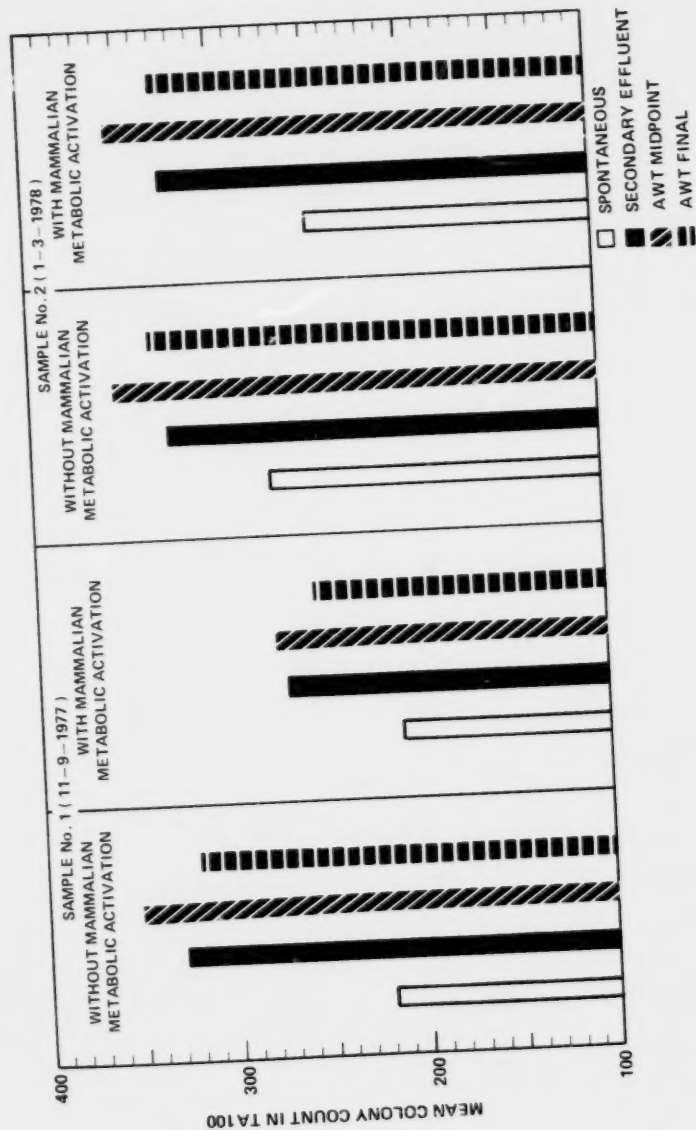


Figure 3. Mutagenic Response of the Wastewaters from Piscataway Model AWT Plant in Salmonella Strain TA 100 (20 ml wastewater/assay). The Salmonella medium was prepared in filter sterilized wastewater.

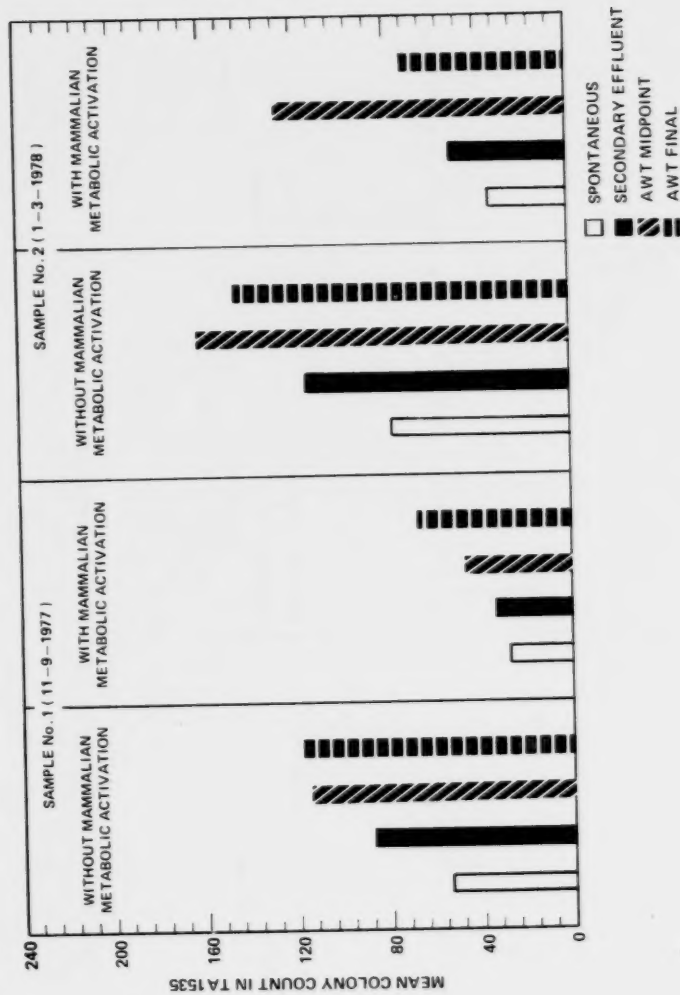


Figure 4. Mutagenic Response of the Wastewaters from Piscataway Model AWT Plant in *Salmonella* Strain TA 1535 (20 ml wastewater/assay). The *Salmonella* medium was prepared in filter sterilized wastewater.

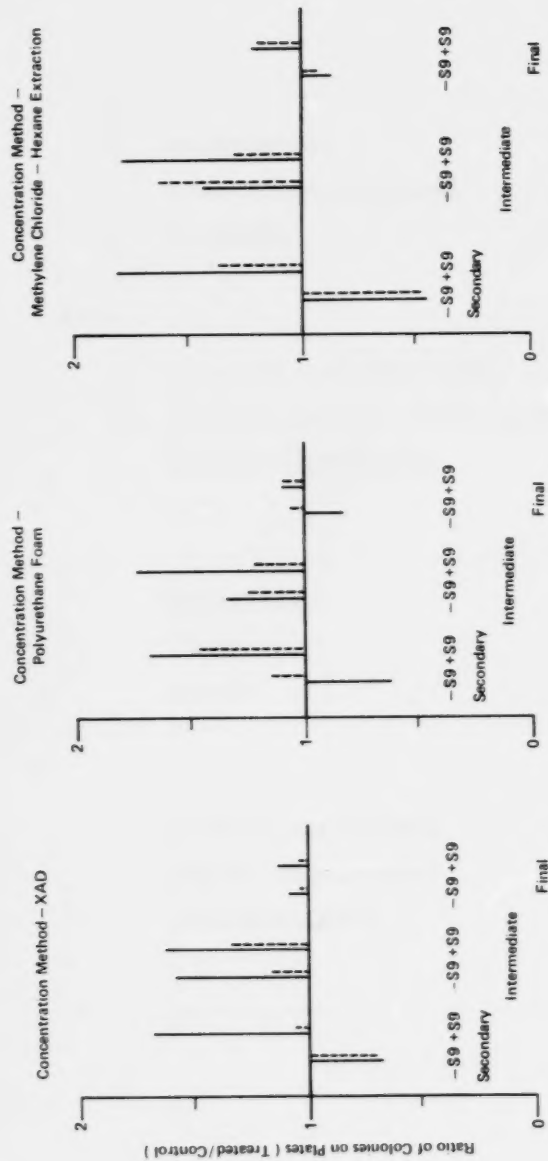


Figure 5. Mutagenic Response of the Organic Mixtures Recovered from Secondary Municipal Effluent after Partial and Full AWT Treatment in *Salmonella* Strain TA 100. Treated/Control Colonies ratio below 1.0 represents toxicity. ----- Concentrate volume equivalent to 20 ml wastewater; ----- Concentrate volume equivalent to 50 ml wastewater.

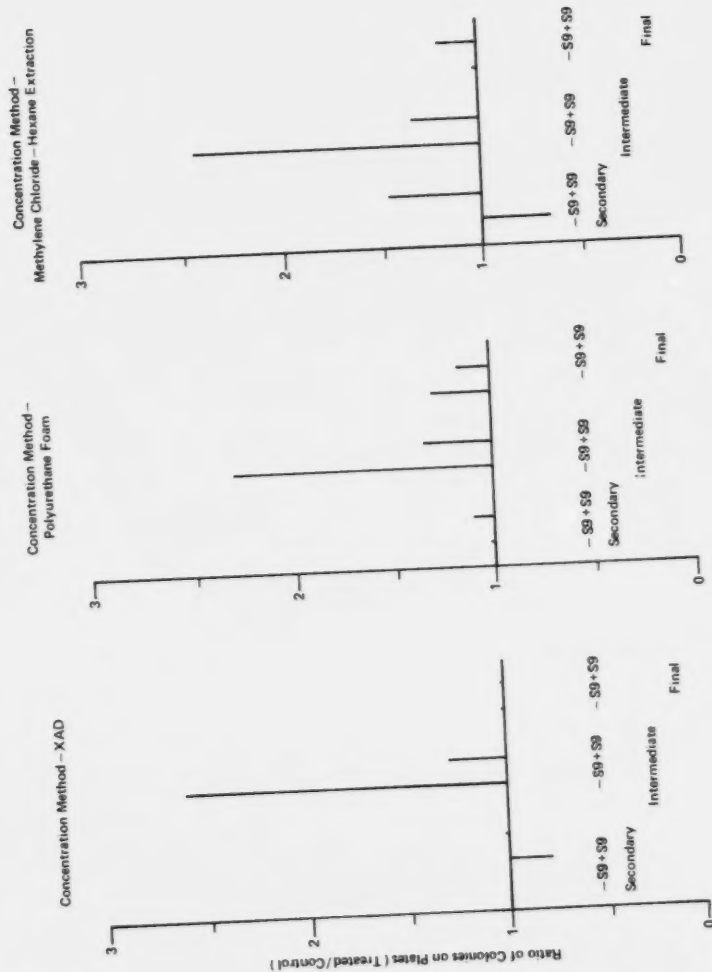


Figure 6. Mutagenic Response of the Organic Mixtures Recovered from Secondary Municipal Effluent and Effluents after Partial and Full AWT Treatment in Salmonella Strain TA 1538. Treated/Control Colonies ratio below 1.0 represents toxicity. The data shown above is for concentrate equivalent to 50 ml wastewater.

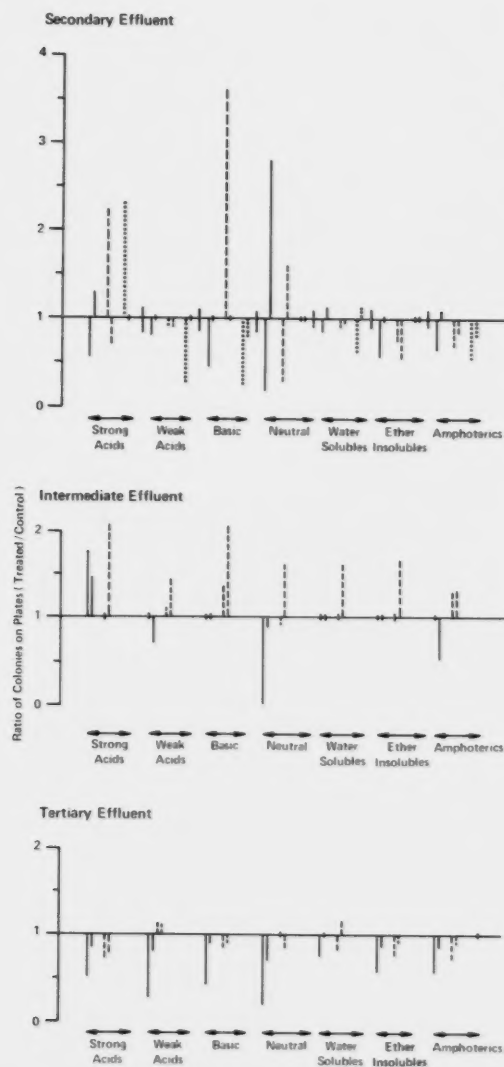


Figure 7. Mutagenic Activity of the Major Classes of Compounds Recovered from Wastewaters with Polyurethane Foam Plugs as Sorbent. — Salmonella Strain TA 98; ---- Strain TA 100; Strain TA 1538. First vertical line with each strain represents mutagenic activity in the absence of mammalian activation, the second vertical line is in the presence. The response shown is for each fraction volume representing 100 ml wastewater. Data not shown for Strain TA 1538 for intermediate and tertiary effluent.

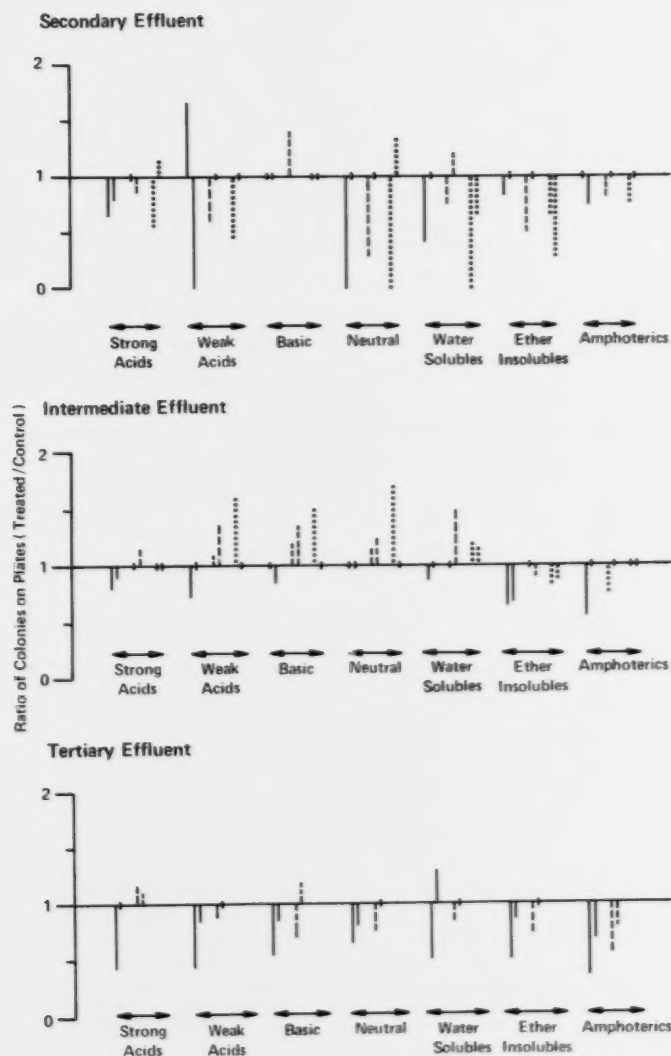


Figure 8. Mutagenic Activity of the Major Classes of Compounds Recovered from Wastewaters with XAD as Sorbent. — *Salmonella* Strain TA 98; --- Strain TA 100; Strain TA 1538. First vertical line for each Strain represents mutagenic activity in the absence of mammalian activation system, the second line in the presence. The response shown is for each fraction volume representing 100 ml wastewater. Data not shown for Strain TA 1538 for intermediate and tertiary effluent.

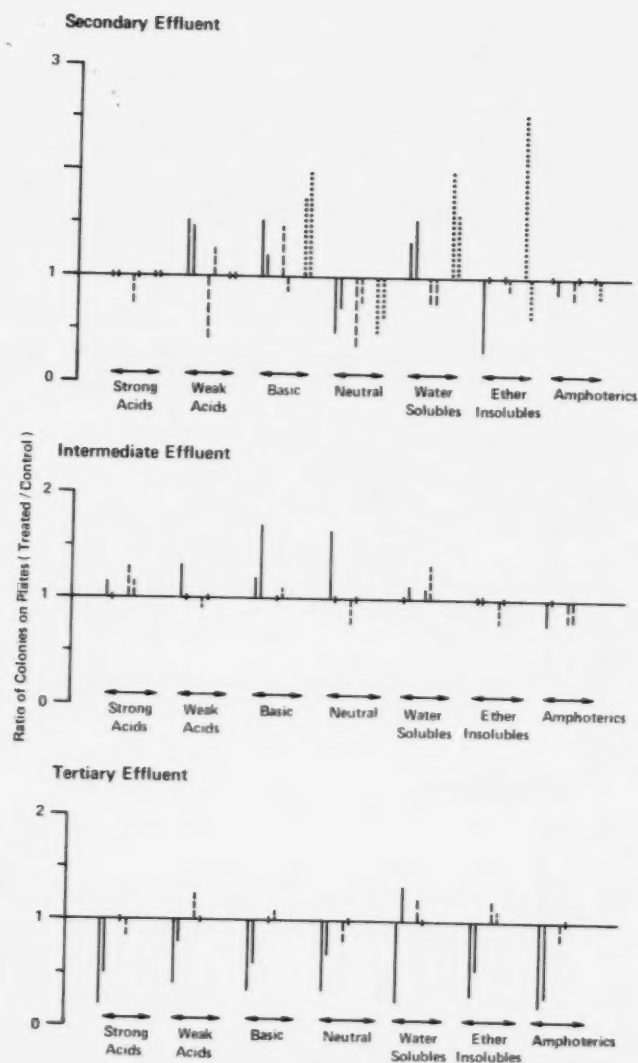


Figure 9. Mutagenic Activity of the Major Classes of Compounds Recovered from Wastewaters by Methylene Chloride-Hexane Extraction. — Salmonella Strain TA 98; ---- Strain TA 100; Strain TA 1538. First vertical line with each Strain represents mutagenic activity in the absence of mammalian activation system; the second line is in the presence. The response shown is for each fraction volume representing 100 ml wastewater. Data not shown for Strain TA 1538 for intermediate and tertiary effluent.

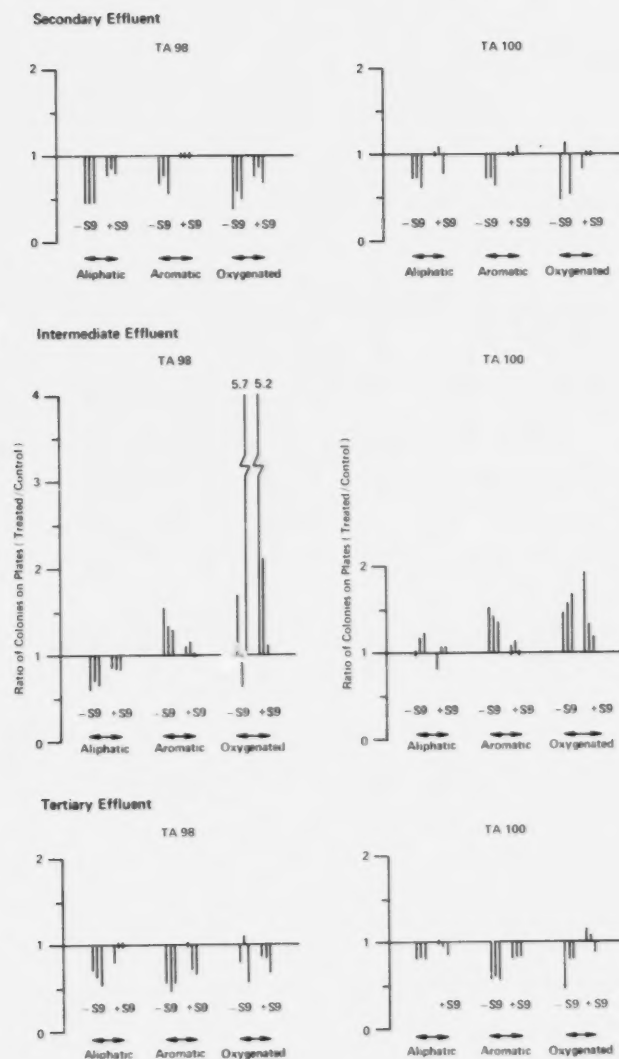


Figure 10. Mutagenic Activity of the Components of Neutral Fraction of Wastewaters Collected with Polyurethane Foam as Sorbent. The three vertical lines in each set represent three different concentrations of the fraction tested. Concentrations from left to right are: Aliphatic representing 200, 100, and 50 ml wastewater; Aromatic representing 200, 100 and 50 ml wastewater; Oxygenated representing 50, 25 and 10 ml wastewater.

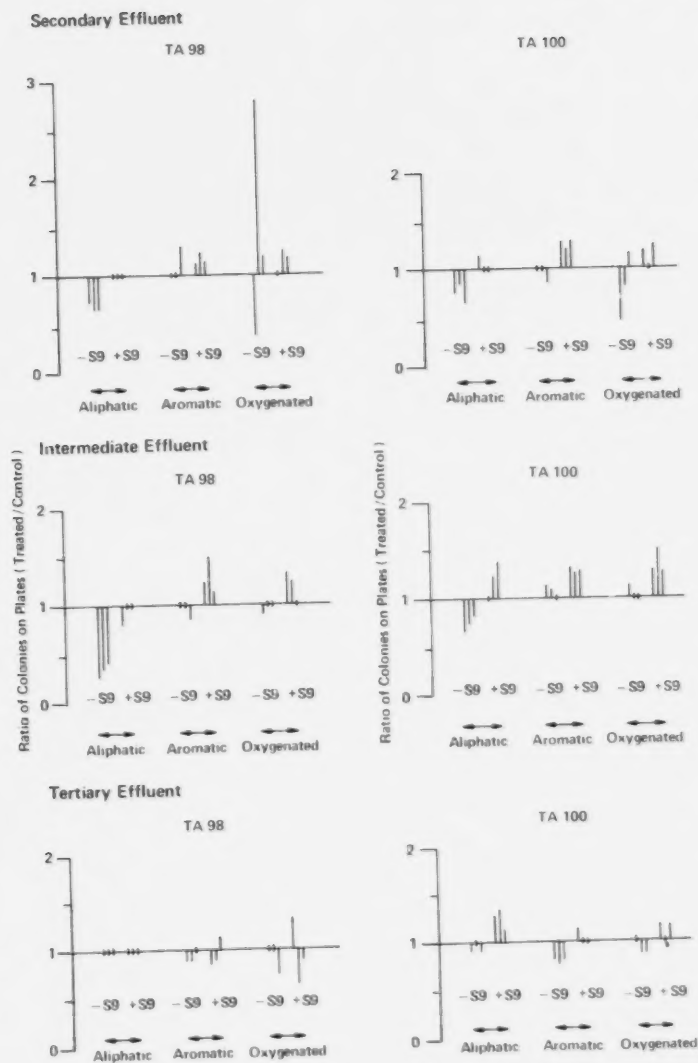


Figure 11. Mutagenic Activity of the Components of Neutral Fraction of Wastewaters Collected with XAD as Sorbent. The three vertical lines in each set represent three different concentrations of the fraction tested. Concentrations from left to right are: Aliphatic representing 200, 100 and 50 ml wastewater; Aromatic representing 100, 50 and 25 ml wastewater; Oxygenated representing 100, 50 and 25 ml wastewater.

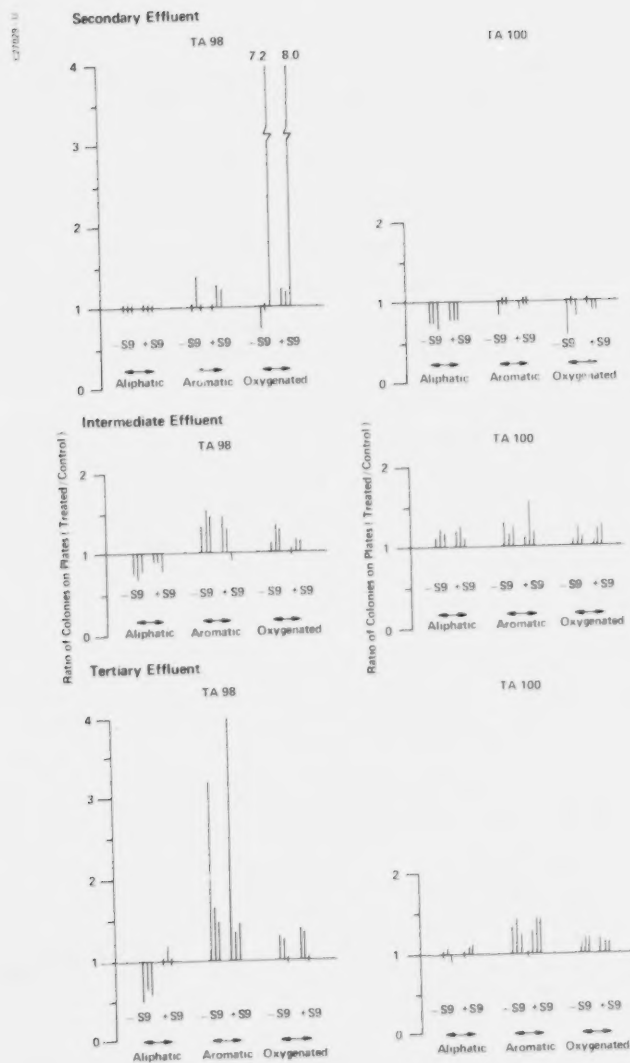


Figure 12. Mutagenic Activity of the Components of Neutral Fraction of Wastewaters Recovered by Extraction with Methylene Chloride-Hexane. The three vertical lines in each set represent three different concentrations of the fraction tested. Concentrations from left to right are: Aliphatic representing 200, 100 and 50 ml wastewater; Aromatic representing 200, 100 and 50 ml wastewater; Oxygenated representing 50, 25 and 10 ml wastewater.

INTEGRATED USE OF BIOASSAYS AND CHEMICAL ANALYSES TO
EVALUATE THE QUALITY OF REUSE WATER

Terence E. Cody, Ph.D.

Victor J. Elia, Ph.D.

C. Scott Clark, Ph.D.

Robert T. Christian, Ph.D.

Department of Environmental Health, University of
Cincinnati Medical Center

There is a pressing need for pure water for drinking, irrigation, and recreation in many geographic areas with rapidly growing populations. This, coupled with rapid progress toward development of efficient and cost-effective water purification processes, is likely to bring public acceptance of direct water reuse sooner than expected. Water purified by recycling systems and intended for human contact or consumption must meet high standards of quality to protect public health. The question of what those standards should be has not yet been satisfactorily resolved, but they will undoubtedly be required to match, perhaps surpass, present U.S. Environmental Protection Agency primary drinking water standards (1). Recent reliability studies on wastewater treatment for reuse conducted by the Environmental Protection Agency have used the primary drinking water standards and current national drinking water quality measurements made during the Environmental Protection Agency National Organics Reconnaissance Survey in 1975 as the benchmarks against which to evaluate their plant (2).

Quality control of potable water supplies, whether drawn from polluted natural sources or produced by recycling wastewater, would benefit from the use of biological screening assays in addition to chemical assays.

Wastewater and natural water are complex mixtures of dissolved and suspended materials. Exhaustive chemical analysis of water samples can not be performed in a reasonable time and at reasonable cost. Furthermore, interpretation of data gathered by chemical analyses of water samples can not be readily interpreted in terms of the toxic effects that may result from consumption of the water. Biological methods for evaluation of potential toxic effects of potable water could effectively supplement chemical analyses.

The method of Ames (3), using strains of *Salmonella typhimurium*, has been widely used for detection of mutagenic substances. This method has been applied to testing recycled water samples (4). General toxicity testing of potable water supplies with rapid biological screening assays has not been as well studied, although we (5) and Gruener (4) have applied such assays to recycled water. This paper presents some examples of the use of a mammalian cell tissue culture assay for assessment of quality of recycled wastewater.

The Bioassay Method

The goal of developing methods of biological monitoring for trace levels of toxic contaminants in treated water is to provide techniques that are rapid and can be used in conjunction with currently used chemical methods for screening or monitoring produced waters for potential health effects. We have developed a bioassay approach that has been very useful in our studies.

The water sample is collected and chilled immediately, but not frozen, and stored at about 4°C until ready for testing. The sample is passed through a 0.22 μ pore size membrane filter for removal of particulate material, including bacteria and viruses. Salts, glucose, amino acids and vitamins are added to the sample to convert it into tissue culture medium. The media to which the cultures are to be exposed must have a nearly neutral pH that does not vary more than a few-tenths of a pH unit during the exposure period. The osmotic pressure of the media must be within the range of 290-310 mmol/kg H₂O. If the osmolality exceeds this range, a condition that may occur when testing water samples with moderately high levels of dissolved solids, the amount of NaCl added to the water sample when making the medium is adjusted to bring the osmolality within the desired range. The medium is re-sterilized by membrane filtration. Sterile fetal bovine serum, 1% vol/vol, and antibiotics are added to the medium.

Two days prior to exposure, cultures of mouse fibroblasts (L-cells, Clone 929) are prepared by seeding culture flasks with 20,000 cells/cm² of surface. The cells attach to the surface of the flask and approximately quadruple in number by the time the growth medium is replaced with medium prepared from test water. Cultures are exposed for 72 hours at 37°C. During the exposure the microscopic changes in the cell cultures are monitored daily. At the end of the exposure period quantitative assessment of the effects of the test water are determined through measurement of protein or DNA contents or cell numbers in the cultures.

Chemical analysis of the water sample is usually performed on the samples before and after filtration and on the medium itself. In our experiments the chemical analyses performed were chosen on the basis of what was known about the sample and usually included some or all of the following: total organic carbon (TOC); chemical oxygen demand (COD); total dissolved solids; atomic absorption analysis for metals; gas chromatographic head space analysis for volatile organics; and high pressure liquid chromatography for non-volatile organics.

The general approach has been adaptable to screening for toxicity for all types of water samples that have been encountered: drinking water; wastewater; recycled water; natural surface and ground waters.

The Response of Tissue Cultures to Toxic Chemicals and Mixtures

By varying the concentration of chemicals or percentage of test water in the medium, the concentration-dependent response of the cultures to the test substance may be determined. The data obtained from exposure of cultures to varying concentrations of test substance (Figures 1 and 2) may be used to determine the concentration of chemical or water sample that will cause a 50% reduction in growth. This value, referred to here as the LC₅₀ (50% lethal concentration), is used to compare the effects of different test substances on the cultures. Comparisons should be made among experiments in which the conditions of exposure are similar. The concentration-dependent responses illustrated in Figures 1 and 2 were a result of a 72-hour exposure. In our experience

a 72-hour period is long enough to observe a response at the lowest effective concentration and yet not so long as to introduce error due to build-up of excessive metabolites and cell degradation products in the medium.

Both of the curves shown are typical of the type of response usually seen when protein or DNA production is used as the estimation of cell growth. The concentration range and the variability of response will vary depending on compound and mixture tested. However, the sharpness of the response of the cultures to the RO permeate is not atypical for complex mixtures.

Comparison of Treated Wastewater With Cincinnati Tapwater

Synthetic hospital wastewater was treated by ultrafiltration, reverse osmosis, and ozonation. This water sample caused no toxic response in the mammalian cell culture assay. In order to determine how the treated water sample compared with the biological quality of drinking water, samples of the treated wastewater and Cincinnati tapwater were concentrated by vacuum fractional distillation. Concentrates of both water samples were obtained that were toxic in the tissue culture assay.

Vacuum fractional distillation was used because water could be removed from the samples by reducing the pressure rather than raising the temperature. The low-boiling fractions were distilled and saved to be recombined with the residue in the stillpot after the water-containing fraction had distilled. The volume of the Cincinnati tap sample was reduced to 0.005 of original (200-fold concentrate) and the TOC was increased from <2 to 77 mg/l. The treated wastewater was reduced in volume from 123 liters to 2.2 liters (~0.02 of original volume; 55-fold concentrate). Based on TOC measurements, which increased from 12 to 417 mg/l, 60% of the organic matter was conserved in the concentrate. The remainder was present in the distillate.

Cincinnati tapwater was toxic when the volume was reduced to <0.05 of original volume (20-fold concentrate). Protein content of cultures treated with water reduced to 0.005 of original volume (200-fold concentrate) for 72-hours was 25% of control protein. Tissue culture protein was reduced 45-55% when treated with water reduced to 0.009 (112-fold) of original volume (Table 1).

The most concentrated sample of treated wastewater (~0.02 of original; 55-fold) and a volume reduction to ~0.04 completely killed the cells in tissue cultures. When the Cincinnati tap was tested at these concentrations it reduced the protein content of cultures by <20% and <10%, respectively. The treated wastewater required a concentration of less than one-half before the first toxic effects were observable. Therefore, the treated wastewater was shown by the concentration procedure to be substantially more toxic than Cincinnati tapwater. More extensively treated wastewaters prepared for later studies proved to be at least as nontoxic as Cincinnati tapwater.

As an alternative to vacuum fractional distillation, freeze concentration has been used with even better retention of organic components. The efficiency of the freeze concentration method declines if the water contains high concentrations of dissolved salts or suspended solids.

Reduction of Toxicity of RO-Treated Wastewater by Ozonation

Cytotoxicity assays may be combined with chromatographic analyses to

evaluate the effectiveness of a wastewater treatment step. In the following experiment, samples of a simulated wastewater that was treated with reverse osmosis were ozonated in batches for increasing periods of times. The changes in chemical composition were followed with high pressure liquid chromatography (HPLC) and the changes in toxicity were followed with the tissue culture assay system. The treatment system that was used is described by Elia et al. (6).

Ozone was generated from an O₂ stream with a Wellsbach O₃ generator at a concentration of 67.5 (65-70) mg O₃/l O₂ at a gas flow rate of 2 std. l/min. The gas stream was introduced through fritted glass spargers into two connected columns containing a total volume of 13 liters of wastewater. The pH of the wastewater was 9.0-9.5 and the temperature was 50-60°C. O₃ input rate was 131 (130-134) mg/min. during all exposure periods except during the 120 min. period when it was 120 mg/min. O₃ was consumed throughout the exposure periods. The rate of consumption became less as the exposure period was lengthened (Table 2).

Chromatographic analysis by HPLC showed that treatment with ozone changed the chemical composition of the treated water (Table 3). The changes were time dependent. An unexpected alteration of the chemical composition occurred when the RO-treated wastewater was treated with O₂ for 15 minutes. The major product formed by the O₂ treatment was not the same as the major product formed by treatment with O₃ in an O₂ carrier stream. Treatment with O₂ for 15 minutes resulted in reduction of the peak with a retention volume of 4.3 ml from >190 to 17 units. At the same time the peak at 2.1 ml increased from 8 to 143 units. In contrast, ozonation eliminated all of the observable peaks in the RO-treated wastewater in <7.5 minutes. New peaks were formed at 2.3 ml, 3.3 ml and 3.7 ml. The peak at 2.3 ml was formed in <7.5 ml and then diminished with increasing O₃ time. The peak at 3.7, after being formed did not change between 7.5 and 15 minutes and, if present, was obscured by the 3.3 ml peak after 30 minutes of ozonation. The major peak in the ozonated samples was the 3.3 ml peak which increased from 50 units after 7.5 minutes of ozonation to 184 units at 30 minutes.

The tissue culture assay response was also influenced by the length of ozonation time (Table 4). The RO-treated waste was lethal in 72 hours. A 15-minutes exposure to ozone did not reduce the toxicity. By 30 minutes the toxicity of the water had been reduced to the extent that it only inhibited protein synthesis by 30-35%. There was no further reduction of toxicity even after 120 minutes of ozonation. During the first 30 minutes of ozonation the COD decreased rapidly from 86 to 10 mg/l. It continued to drop slowly to <3.5 mg/l by 120 minutes of ozonation.

In fifteen minutes of purging with oxygen the toxicity of the water was reduced to the point that it only inhibited protein production by about 50-55%. Concomitantly, the COD of the solution was reduced from 86 to 75 and alteration of chemical composition occurred.

The alteration in chemical composition during oxygen purging without a great reduction in COD suggests that reduction in toxicity occurred as a result of alteration rather than substantial degradation of toxic components. In the ozonation experiments, the initial reduction in toxicity may also have resulted from alteration of toxic components but nearly complete removal of COD also occurred in the first 30 minutes of ozone treatment which surely contributed to removal of contaminants that caused the toxic response in tissue cultures.

Acknowledgements

This research was supported by the U.S. Army Medical Research and Development Command, Washington, D.C., under contract No. DADA-17-73-C-3013 and by National Institutes of Environmental Health Sciences Center Grant ES-00159. We also gratefully acknowledge the technical assistance of Mr. Kevin T. McGinnis and Ms. Mary Earley.

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TABLE 1
Cytotoxicity of Treated Waters

	TOC mg/l	Inhibition of Protein Synthesis
Treated Synthetic Waste	12	None
55-fold concentrate	417	100%
28-fold concentrate	208	100%
9-fold concentrate	67	50-60%
Distillate	5	None
Cincinnati Tapwater	<2	None
200-fold concentrate	77	75%
112-fold concentrate	43	45-55%
64-fold concentrate	25	20-30%
20-fold concentrate	8	0-10%

Simulated hospital waste was treated with UF, RO, and ozone.
Concentration was by vacuum fractional distillation.

TABLE 2
Consumption of O₃ by RO-Treated Simulated Wastewater*

O ₃ Treatment min	O ₃ Input Rate mg/min	O ₃ Consumption mg/l	O ₃ Consumption mg/l/min
7.5	130	66	8.8
15	134	129	8.6
30	132	240	8.0
45	130	317	7.0
60	130	399	6.6
120	120	671	5.6

*13-1 batches of wastewater at 50-60°C and pH 9-9.5.

TABLE 3
Relative Heights of Peaks From HPLC of Simulated Wastewater

Treatments	Retention Volumes of Peaks (ml)							
	2.1	2.3	2.9	3.3	3.7	4.3	4.5	5.5
Ozonation (min)								
0	8	-	20	-	-	>190	+	4
7.5	-	38	-	50	21	-	-	-
15	-	21	-	72	25	-	-	-
30	-	6	-	184	*	-	-	-
Oxygen (15 min)	143	-	38	-	18	8	-	4

- = Not Detected; + = Present; * = If present, obscured by 3.3 ml peak.

TABLE 4
Reduction of Toxicity and COD Resulting From Ozonation
of RO-Treated Simulated Wastewater

Treatments	COD mg/l	Inhibition of Protein Synthesis <u>%</u>
Ozonation (<u>min</u>)		
0	86	80
7.5	50	90
15	36	90
30	10	35
45	5	31
60	<3.5	36
120	<u><3.5</u>	29
Oxygen (15 min)	75	55

RESPONSE OF L-CELLS TO VARYING CONCENTRATIONS OF HYDROQUINONE

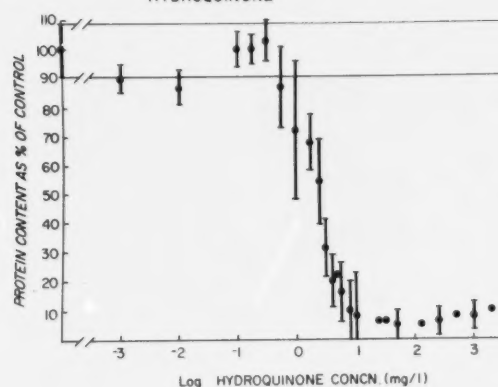


Figure 1. Example of a concentration-response curve of L-cells treated with medium containing a single organic toxic chemical (hydroquinone). The vertical bars through the points designate the limits of $\pm$ std. deviation.

EFFECT OF REVERSE OSMOSIS PERMEATE ON L - CELLS

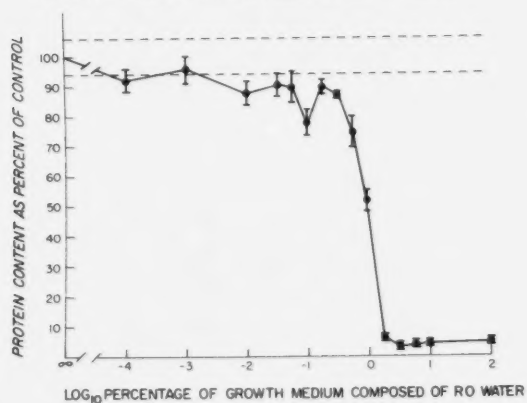
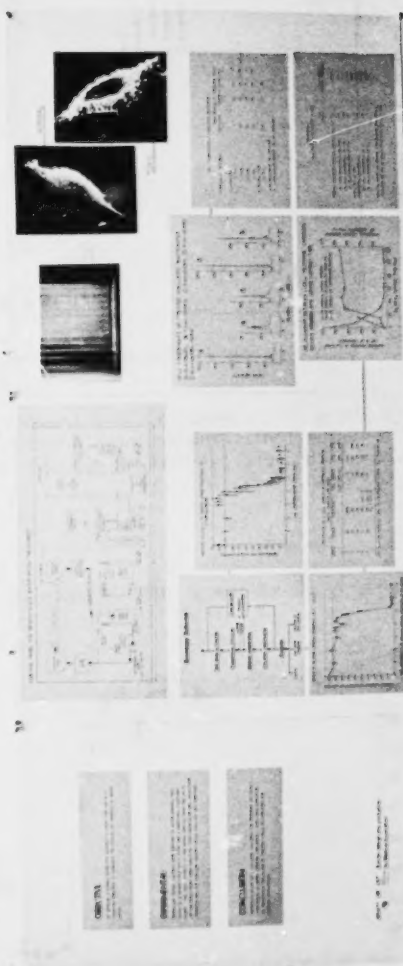


Figure 2. Example of a concentration-response curve of L-cells treated with medium containing a treated wastewater sample. Vertical bars through the points designate limits of $\pm$ std. deviation.

BIOASSAYS & CHEMICAL ANALYSIS FOR EVALUATION OF QUALITY OF RIVER WATER

H. Gade, V. H. S. C. S. Clark, R. T. Chinnam
Department of Environmental Health
University of Georgia



CHEMICAL ANALYSIS AND TOXICITY OF NONVOLATILE ORGANIC COMPOUNDS IN WASTEWATER

Harold F. Walton
Chemistry Department, Box 215, University of Colorado,
Boulder, Colorado 80309

Organic compounds in filtered wastewater are concentrated by pumping the water through a column packed with the absorbent Bondapak-C₁₈. This is a superficially porous silica of particle size 37-75 microns which carries a chemically bonded layer of octadecyl groups. The material has the absorbent properties of kerosene. It retains less polar organic compounds while letting pass inorganic salts and ionized compounds like salts of fatty acids, as well as highly polar water-soluble compounds like sugars, amino acids and urea.

To remove the absorbed compounds we first flush the column with distilled water, then pass water mixed with methanol according to a linear gradient, starting with pure water and ending with pure methanol 30 minutes later. We continue to pass methanol until all absorbed material is removed. Ultraviolet absorbance is used to detect the organic compounds; the absorbance is recorded on a strip chart. The chart record provides some information about the composition of the organic material, and additional information is obtained by collecting and examining fractions of the solution flowing from the column.

EXPERIMENTAL PROCEDURE

Wastewater is collected on the site in sterile one-gallon bottles. Generally, chlorinated, secondarily-treated effluent is taken just above the point of discharge into the river. It is brought to the laboratory and heated to 60°, then filtered through glass fiber filter paper, first the relatively coarse Whatman GF/A, then Whatman GF/F, which retains 0.7-micron particles. Filtering is done under suction, thus removing dissolved air and carbon dioxide. The pH rises to above 8, and is brought back to 7.5 by adding nitric acid.

Ten to twelve liters of filtered, degassed wastewater are now pumped through a stainless-steel column, 1 cm x 50 cm, packed with Bondapak-C₁₈, using the arrangement shown in Fig. 1. In an alternative procedure, unfiltered wastewater is pumped through a series of sand and glass filters and through a short "collector column" of Bondapak-C₁₈, which is later attached to the inlet of the column¹⁸ shown in Fig. 1. The flow rate during this loading step is 7 ml per minute.

When sufficient wastewater has been passed, the flow is reduced to 5 ml/min, the inlet valve of Pump A (Fig. 1) is turned and the column is flushed with distilled water for 15 minutes. Then the linear water-to-methanol gradient is

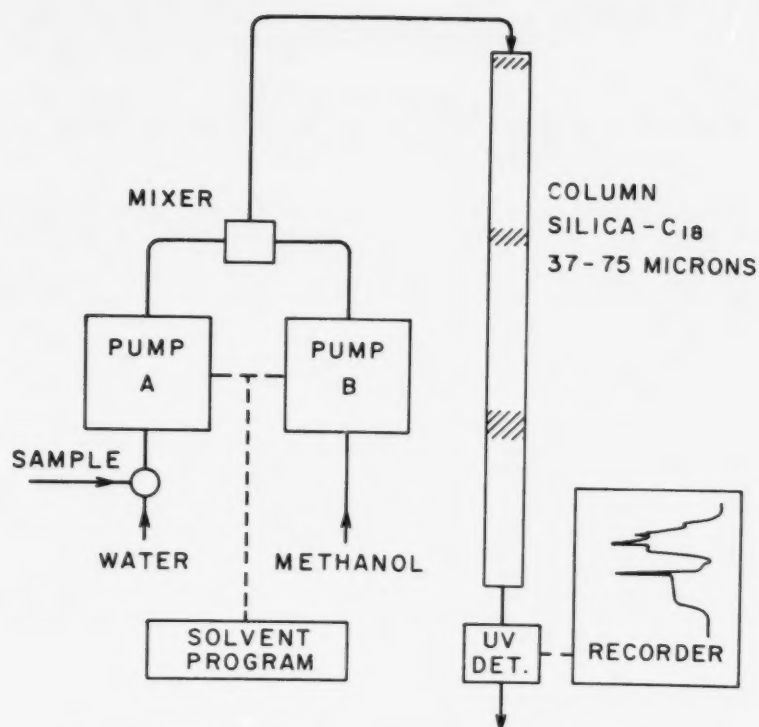


Figure 1: Apparatus for pumping sample and gradient.

Pumps are Model 6000A, controlled by Model 660 solvent programmer, all from Waters Associates; detector was either a single-wavelength model (Chromatronix Inc.) or a variable wavelength detector (Schoeffel Instrument Corp. Model SF770)

started. A schematic graph of absorbance vs. time is shown in Fig. 2, which also indicates where the fractions were taken. Fig. 3 shows chromatograms of water supplied by the San Jose Creek Water Quality Laboratory near Pomona, California; they show the effect of carbon treatment of wastewater, which is to remove the less polar fractions of organic material while having little effect on the more polar fractions. Since the least polar fractions are the most toxic (see below), this information is of some value. Other chromatograms that we have obtained show the effect of carbon treatment and reverse osmosis on Denver wastewater. Reverse osmosis removes ionic and highly polar impurities from water but lets some of the weakly polar fractions pass.

For further examination, fractions of the effluent from the Bondapak column were collected according to the scheme shown in Fig. 2, and were evaporated in a Büchi rotary evaporator under low pressure at 60 -70 C. to a final volume of 2-3 ml.

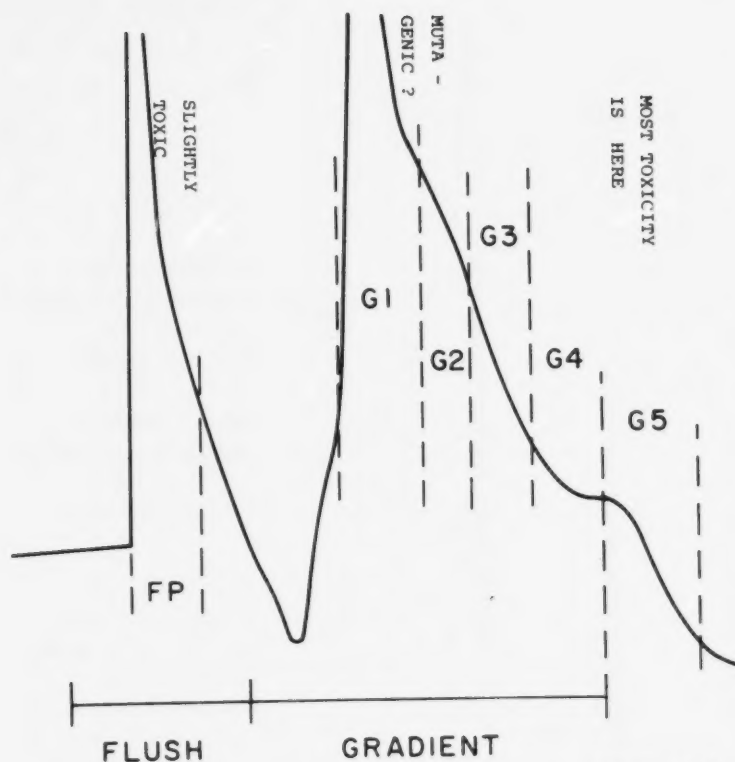


Figure 2: Ultraviolet absorbance record (schematic) and scheme of fraction collection.

A general description of each fraction follows.

The "flush peak", FP, was light brown. It appeared to contain sodium salts of aromatic acids like benzoic acid, along with neutral material. No flush peak appeared if a salt solution was used to flush the column instead of distilled water.

Fraction G1 (G=gradient) was dark brown. Fraction G2 was brilliant green; G3 was brown, and G4 light brown. G5 was light rose pink. In one experiment, each fraction was evaporated in a nitrogen stream at room temperature and the residues were weighed. The weights obtained from one liter of wastewater were: G1, 1.5 mg; G2, 0.6 mg; G3, 1.5 mg; G4, 0.7 mg; G5, 1.0 mg. Assuming that carbon accounted for one-half of the weight (a fair assumption for humic acid), the sum total of fractions G1 through G5 corresponds to 2.7 ppm dissolved organic carbon. This figure is in fair agreement with total organic carbon measurements made on the filtered

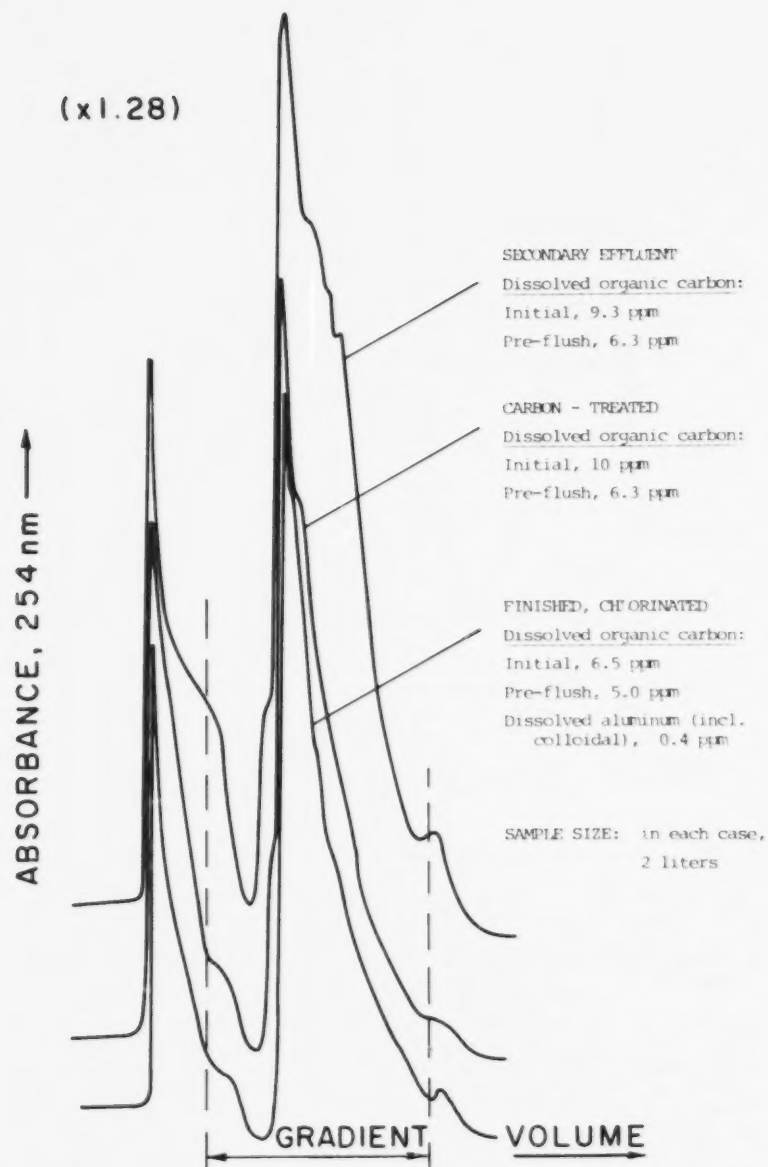


Figure 3: Chromatograms of secondary sewage effluent before further treatment (top curve) and after carbon and chlorine treatments (lower two curves). Samples supplied by the San Jose Creek Water Quality Laboratory.

wastewater before and after passing through the Bondapak column (see the DOC data noted in Fig. 3).

The brown color was due to humic substances of high molecular weight. Gel permeation chromatography of fractions G1 through G4 showed the presence of material with molecular weight 2000 and above, and some of the material was retained on a Diaflo ultrafiltration membrane. The presence of humic material greatly complicates the detection of substances of lower molecular weight. Fraction G5 was almost free of humic material, and we have had some success in identifying the substances that it contains.

Fraction G2 is distinctive for its green color. It also contains a brown component. The green and brown subfractions may be separated by treating the dry residue with pure methanol (the green dissolves, the brown does not) or by gel permeation chromatography (the brown subfraction has a high molecular weight).

TOXICITY AND MUTAGENICITY TESTS

The nature of these tests is described by Clive Solomons in another paper in this Symposium. Toxicity is measured by the sample's effect on the ability of human blood platelets to synthesize ATP. The flush peak showed moderate toxicity, but the highly polar material that passed through the Bondapak column was apparently nontoxic, as were the fractions G1, G2 and G3. Fractions G4 and G5 were toxic, especially G5.

Mutagenicity was measured by the Ames test (performed by Dr. Elias Balbinder). The results were not consistent from one wastewater collection to another, but some of the G1 and G2 fractions were highly mutagenic. The lack of consistency may be due to subtle variations in the wastewater composition from one collection time to the next.

SECONDARY LIQUID CHROMATOGRAPHY: CHEMICAL ANALYSIS OF THE FRACTIONS FROM THE BONDAPAK-C₁₈ COLUMN.

As we have noted, the fractions collected according to the scheme of Fig. 2 were further resolved by chromatography on other columns. The fractions were evaporated to small volume as described above, then injected into the high-resolution columns by means of a sample loop.

The flush peak was resolved on a column of anion-exchange resin of small particle size, using ammonium acetate buffers as eluents. By changing the pH and the concentration of ammonium acetate we could determine whether a particular peak was due to a neutral species or an anion, and if it was an anion, whether its charge was -1 or -2. At least one peak was due to a doubly-charged anion, but we have not yet found its chemical identity.

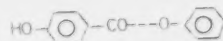
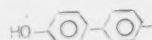
Fractions G1 - G4 were run on columns of porous polymer gels that separate compounds partly by size exclusion, partly

partition. Compounds of high molecular weight (humic acids) were seen to predominate.

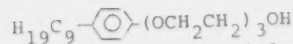
Fraction G5 was run on a MicroBondapak-C₁₈ column (Waters Associates) with 60% methanol for 20 minutes, then a 30-minute linear gradient to 100% methanol. Fig. 4 shows a typical chromatogram. Many peaks are seen, showing the complexity of the material. A fluorescence detector, placed after the variable-wavelength UV detector, showed three strong peaks, two of them in places where no ultraviolet absorption was seen at the wavelength chosen.

Peaks "E" and "F", Fig. 4, were collected and examined. The ultraviolet absorption spectrum of each peak was scanned and the solutions were evaporated. The weights of the residues corresponded to initial concentrations in the original wastewater of 100 µg/l (100 ppb) and 150 µg/l, respectively. Portions of the residues, which were liquids, were introduced into a Hewlett-Packard Model 5980A mass spectrometer by means of a probe, and the mass spectra were scanned and stored as the probe was heated.

Fraction "E" was pink; the pink color appeared at the detector outlet just before the UV absorption started to rise. As the solution evaporated a bright red solid speck appeared. In the final residue the mass spectrometer showed three compounds, the first being tri(butoxyethyl)phosphate, a common plasticizer, and the second, dioctyl phthalate. The third mass spectrum, which appeared last as the probe was heated, was not identified, but seemed to be a phthalate ester. Phthalate esters show ultraviolet absorption maxima near 274 nm; fraction "E" showed a maximum at 262 nm. The UV absorption must have been due to another component, not seen in the mass spectra, and having a high absorptivity. From the "Atlas of Spectral Data" (J. G. Grasselli, editor; CRC Press) likely candidates are:



Fraction "F" showed a clear mass spectrum of a nonylphenol ether, probably



but, again, the ultraviolet absorption spectrum was that of another compound with a maximum at 275 nm. Probably this was a phthalate ester.

Thus fraction "E" contained at least five compounds, and fraction "F" at least two. The complexity of the analytical problem is evident. Mass spectrometry is the most powerful way of identification, but if a compound does not volatilize without decomposition it will not give a mass spectrum. Another tool of identification is infrared spectrometry. We intend to improve our infrared technique to handle the very small samples that are available.

One route to peak identification in chromatography is to match the observed retention volumes with those of known compounds. Several known compounds were run on the MicroBondapak-C₁₈ column under the same conditions as G5 and the retention volumes were noted. The toxic pollutants

FRACTION G5

270 nm, 0.1 AUFS

μ BONDAPAK-C₁₈

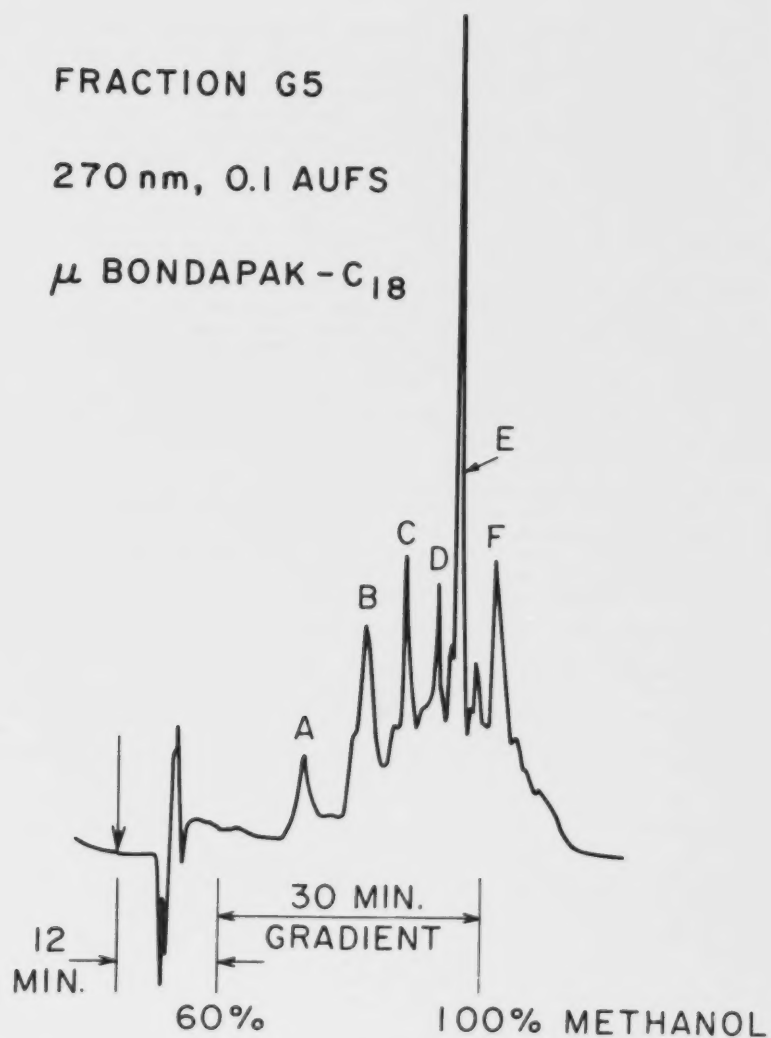


Figure 4: Chromatogram of fraction G5 on MicroBondapak-C₁₈. In this case a 12-minute isocratic flow of 60% methanol was used, followed by a 30-minute linear gradient to 100% methanol. Later a 20-minute flow with 60% methanol was adopted.

benzidine, p-nitrophenol, nitrobenzene, m-chlorophenol and atrazine all came out before peak "A" of Fig. 4, and we concluded that these pollutants were absent from Denver secondary sewage effluent. Polychlorinated biphenyls eluted in the region of peaks B to F; however, the mass spectra of compounds containing chlorine are very characteristic, because of the chlorine isotope pair, and the mass spectra of E and F gave no sign of chlorine atoms.

CONCLUSIONS

Most nonvolatile organic matter in secondary sewage effluent consists of highly polar, water-soluble compounds and humic material. About one-quarter of the dissolved organic carbon can be retained on an octadecyl-silica reverse-phase chromatographic absorbent, Bondapak-C₁₈, and it can be completely removed from this absorbent by stripping with a water-to-methanol gradient. Fractions have been collected during the stripping gradient and examined for toxicity, mutagenicity and chemical composition.

Most of the material absorbed on the Bondapak-C₁₈ consists of humic substances that are apparently nontoxic, though they may have associated with them some substances that show mutagenic activity. The toxic compounds seem to be weakly polar or nonpolar, and to amount in all to less than one milligram per liter of secondary effluent.

High-resolution liquid chromatography combined with mass spectrometry shows the presence of many compounds whose concentrations are in the low parts-per-billion range. Identification is difficult, but by comparing observed chromatograms with the chromatograms of known substances, we may say that certain toxic pollutants, such as atrazine, chloro- and nitrophenols and benzidine are not present.

ACKNOWLEDGEMENTS

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AEROSOL GENERATION IN SPRINKLER IRRIGATION

Bahman Sheikh-ol-Eslami, Phillip J. Morris, Joyce S. Hsiao
Project Managers
Engineering-Science
600 Bancroft Way
Berkeley, California

INTRODUCTION

The Monterey Wastewater Reclamation Study for Agriculture (MWRSA) consists of a number of studies examining irrigation with reclaimed wastewater to answer the growing concerns of the farming community. One ancillary study of MWRSA aims to determine the nature and extent of aerosols generated during sprinkler irrigation and compares use of reclaimed wastewater with normal well water. The chief objective is to assess the risk to human health when in contact with aerosols generated with reclaimed wastewater. Results of the study will be used to establish safe buffer zones around the experimental and demonstration sites.

APPROACH

The approach employs three steps. First, an optical scanning device is used to count the number of particles in various size ranges from one to 100 μm at various distances from the sprinkler line. Second, a chemical tracer, lithium chloride, is being used in the irrigation water to distinguish sprinkler-generated aerosols from background aerosols. Third, Anderson Samplers are used to assay airborne viable biological contaminants in aerosols.

RESULTS AND DISCUSSION

Background atmospheric and spray irrigation aerosols were measured in the Castroville area during September 1978. The results indicate that background concentrations are strongly influenced by wind speed and relative humidity. Sampler efficiency also varied with wind speed in a detectable, but not yet fully understood manner. Background aerosol concentrations averaged about 3.1×10^7 particles per cubic metre and varied by a factor of at least two. By number, far less than one percent (30 to 35 percent by mass) of the aerosol particles were greater than 10 μm in diameter. Measurements 15 m downwind of a line of sprinklers during a 2.5 m/s wind indicated a concentration of 3.4×10^7 particles per cubic metre. However, nearly one percent of the particles by count and 50 to 55 percent by mass were greater than 10 μm in diameter.

Inhalation and Ingestion of Aerosols

The human nose can be considered as a sampling device whose efficiency depends upon wind speed, orientation to the wind and breathing rate. For random orientation in wind speeds from 0.75 to 2.75 m/s and a somewhat accelerated breathing rate of 20 litres per minute, capture efficiencies may be expected to vary from about 50 percent at 10 μm to 100 percent of the very-small-diameter particles (0.5 μm) (Reference 1). Further, data indicate the fraction inhaled decreases to about 30 percent for particles with 30 μm diameters (aerodynamic diameter).

Particle size also determines where inhaled particles are deposited. Above 10 μm , nearly all particles are deposited in the nasopharyngeal region, where (with normal processes) a portion reaches the stomach. For particle size 1 μm or smaller, virtually none are deposited in this region. Particle deposition in the pulmonary system increases from about 10 percent for 10- μm particles to 30 percent for 0.2- μm particles. A very small fraction of inhaled particles in the 0.1 - 100- μm range is deposited in the tracheobronchial region and thence to the stomach (Reference 2). Thus, aerosols containing virus have a fairly high chance of being deposited in the pulmonary system as they can be of much smaller diameters. Those containing bacteria, on the other hand, are necessarily too large to reach the lungs and would primarily be deposited, ultimately, in the stomach.

Risk Analysis

A major factor influencing risk is the concentration of pathogenic organisms in sprayed effluent. Pathogen concentrations as high as 10^2 or 10^3 pathogens per litre would likely result in an unacceptable risk from aerosol inhalation or ingestion. However, with an assumed, projected pathogen concentration of seven per litre in the filtered secondary effluent, the probability of inhaling a viable pathogen during an eight-hour exposure at a distance of 15 m (50 ft) from the sprinklers is computed to be approximately one in 86,000.

To arrive at this probability, the following procedure and assumptions were employed. From the results of field measurements of aerosol counts in the various particle size ranges, it was computed that 0.86×10^{-6} litres of aerosols in one cubic metre of air are contributed by the sprinklers at a distance of 15 m from the sprinkler line. Even though some pathogens are deactivated by the shock of aerosolization and other phenomena, 100 percent survival is assumed as a conservative measure of night-time conditions. Assuming that the human nose captures 50 percent of the particles with a random orientation; that a wind speed of 0.75 to 2.75 m/s prevails; that a breathing rate of 20 per minute (4.8 m^3 of air in eight hours) occurs; and that 80 percent of the aerosols are deposited in the nasopharyngeal regions, the probability of inhaling one pathogen during an eight-hour exposure is, conservatively:

$$P = 0.86 \times 10^{-6} \times 7 \times 0.50 \times 0.80 \times 4.8 = 1.16 \times 10^{-5}$$

or, 1 in 86,000.

The risk of becoming ill as a result of such exposure depends on "dose-response" relationships of the specific pathogen and the condition of health of the exposed person. For example, if it takes 10,000 pathogens before an individual becomes clinically sick from inhalation of microorganisms in the aerosol, then the risk of exposure will become one in 860,000,000.

Field research is continuing so that the various relevant factors affecting risk can be refined. However, the focus of MWRSA is on assuring a consistently high quality of effluent and on avoiding upset conditions which would adversely affect pathogen concentrations in the effluent.

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EVALUATION OF HEALTH EFFECTS DATA ON THE REUSE OF SHOWER AND LAUNDRY WATERS BY FIELD ARMY UNITS

David Cogley, Wesley Foy, William Light,
Walden Division of Abcor, Inc.

James C. Eaton, Jr.
U. S. Army Medical Bioengineering Research and
Development Laboratory

Marcus Mason
Consultant to Walden Division of Abcor, Inc.

INTRODUCTION

The feasibility of short-term shower and laundry water reuse by field Army units has been evaluated for the aspect of health effects. Four topics have been investigated:

- (1) the composition of shower and laundry wastewaters as a function of the health care products employed;
- (2) an engineering evaluation of the treatability of these health care product ingredients and their likely transformation products in wastewater treatment systems;
- (3) oral, dermal and ocular toxicity of these ingredients and transformation products; and,
- (4) previous Army-sponsored research related to shower and laundry water reuse.

None of the available data indicate that a toxic response is to be expected for soldiers using treated, recycled shower and laundry waters. The next scientific investigation to be considered is a human clinical trial of shower and laundry water reuse. Such a study could be performed only in compliance with U. S. Army Medical Research and Development Command Regulation 70-25, "Use of Human Subjects in Research, Development, Test, and Evaluation." This regulation requires: a valid scientific justification, measures to minimize risk to test subjects, and administrative review procedures. To satisfy the regulation, it is recommended that consideration be given to the preparation of several assessments, criteria, and protocols.

SHOWER AND LAUNDRY WATER COMPOSITION

The composition of shower and laundry wastewaters was estimated by determining the constituents of cleansing and health care products. Information was obtained through telephone contacts, computerized data base searches, and assessment of books and printed reports.

Telephone contacts included: Food and Drug Administration (FDA); Consumer Product Safety Commission (CPSC); Soap & Detergent Assoc.; Cosmetic, Toiletry, and Fragrance Assoc.; United States Environmental Protection Agency (USEPA); USEPA NERC, Cincinnati; International Joint Commission on Health Effects of Non-Phosphate Detergents; Union Carbide Corp.; NASA Johnson Space Flight Center; NASA Langley Research Center; NASA Ames Research Center; Chemicals Industry Institute of Toxicology; and U.S. Army Natick Research & Development Command.

Computerized data bases were searched with customized key word lists. Data bases searched were Chemical Data Base (Chemical Abstracts), NTIS, TOXLINE, Chemical Abstracts, NTIS Government Reports Announcements and Index, DDC Technical Report Program-Past Research, DDC Research and Technology Work Unit Information System, Environment Index, Pollution Abstracts, Biological Abstracts, Engineering Index, Energy Index, Nuclear Science Abstracts, Atomindex, Selected Water Resources Abstracts, and Energy Info Data Base.

These searches resulted in the acquisition of approximately 30 journal articles and the books and reports listed in the bibliography to the present report.

Calculations of wastewater component concentrations were based on USAMBRDL-supplied lists (Tables 1 and 2) of products present in shower and laundry wastewater and the concentration of each type after a single use of the water. For each of the product types noted, composition data were obtained for several types or brands of products. Constituent concentrations were calculated for each product, corresponding to product concentrations. The highest concentration of each constituent was selected for each product type, and these were summed by product. Final concentrations of constituents in wastewaters were calculated for shower wastewaters, laundry wastewaters, and a mixture of the two. Ingredient concentrations for laundry, shower, and mixed (55% shower, 45% laundry) wastewater are given in Tables 3, 4, and 5.

TREATABILITY

Four wastewater systems were evaluated for treating shower and laundry wastewaters generated by field Army units. Both the components of wastewater identified above and chemicals created during wastewater treatment were considered. Relative system performances at 70-90% water conversion were estimated from calculations of the concentrations of nine challenge contaminants present in treated wastewater, which has been recycled an infinite number of times. The challenge contaminants consisted of six organic chemicals rank-ordered from the list of total contaminants based on high concentration, low removal efficiency, and high toxicity; and three inorganic chemicals selected from the list of total contaminants based on high concentration.

UF-RO System: This system consists of an ultrafiltration (UF) unit for removing suspended solids followed by a reverse osmosis (RO) unit for removing dissolved solutes. On the basis of studies with shower and laundry wastewaters, it was estimated that the dissolved solids removal for UF would be 40% of that for RO. The rejection of dissolved species by UF is primarily a result of the adsorption of these species onto suspended particulates.

UF System: This system consists of a UF unit containing membranes having a smaller porosity (i.e., they are "tight") than the UF membranes which would be included in the UF-RO system. On the basis of results of studies with "tight" UF membranes, it was estimated that the rejection of all contaminants by the UF system would be 50% of that for an RO unit.

Pretreat-ACA-IX System: This system consists of a suspended solids removal, pretreatment unit (e.g., diatomaceous earth filter (DEF) or dissolved air flotation (DAF) followed by an activated carbon adsorption (ACA) unit to remove dissolved organic chemicals and an ion exchange (IX) unit to remove dissolved inorganic chemicals.

ERDLator System: The ERDLator is a treatment system developed by the U.S. Army which is used primarily to remove suspended materials from wastewater. For the present application, the system configuration consists of powdered carbon plus polyelectrolyte addition followed by the ERDLator unit followed by hypochlorination and a pressure diatomite filter. On such a unit for total organic carbon, hexane solubles, detergents, salts, and suspended solids, it was estimated that the rejections for individual chemical contaminants would be 75% of those for ACA.

By applying the simplifying assumptions, the removal efficiencies for the wastewater treatment systems (RE_s) can be expressed in terms of the reverse osmosis rejection (R), the activated carbon adsorption removal efficiency (A), and the combined effect of activated carbon adsorption removal for organics and ion exchange removal (assumed equal to RO) for inorganics (AR). The expressions for RE_s are given as follows:

$$\text{In general, } RE_s = \frac{C_I - C_0}{C_I} = 1 - \frac{C_0}{C_I}$$

where C_I = initial concentration of chemical contaminant in wastewater

C_0 = final concentration of chemical contaminant in treated effluent.

For the UF-RO system, for example,

$$RE_{S1} = 1 - [(1-0.4R)(1-R)] = 1.4R - 0.4R^2$$

For the UF system,

$$RE_{S2} = 1 - [(1-0.5R)] = 0.5R$$

For the Pretreat-ACA-IX system,

$$RE_{S3} = 1 - [(1-0.3R)(1-AR)] = 0.3R + AR - 0.3R(AR)$$

For the ERDLator System,

$$RE_{S4} = 1 - [(1-0.75A)] = 0.75A$$

The concentrations of challenge contaminants in recycled, treated shower and laundry wastewaters were calculated using a recycle model. For the four systems, y (the fraction of unremoved chemical contaminants in each cycle) can be expressed as a function of the system removal efficiency (RE_S) and the conversion (z):

$$y = (1 - RE_S)z.$$

For each contaminant, the concentration after an infinite number of recycles is:

$$X = X_1 = 1 + \frac{y}{1-y}$$

where X_1 is the initial concentration. Results for 80% recycle are given in Table 6.

TOXICOLOGY

Toxicity data were sought for each of the compounds predicted to be present in shower or laundry waters. Oral, dermal and ocular toxicity were considered. The factors most strongly influencing the toxicity of particular compounds were assumed to be: inherent chemical reactivity, chemical formulation (e.g., powder, suspension, emulsion, solution), transformation products (e.g., products of chlorination), concentration, and exposure time.

References listed in the bibliography were consulted for toxicity data. Data were assembled on a compound-by-compound basis. Notations of test compound concentration, test duration, etc. were made.

Ocular toxicity data were found to be lacking for all of the following substances. A single asterisk (*) indicates oral toxicity data lacking; a double asterisk(**) dermal toxicity data lacking: Aluminum chloride, Aluminum formate, Aluminum hydroxide, Aluminum sulfate, Bentonite, Castor oil, Cetyl alcohol*, Corn starch, Dicalcium phosphate, Ethoxylated lauryl alcohol* **, Glycerol monostearate* **, Isopropyl myristate, Isopropyl palmitate*, Lanolin, Magnesium carbonate, Magnesium oxide, Methyl paraben, Potassium oleate, Polyethylene sorbitol monostearate, Propoxylated PABA* **, Sodium-4-chloro-2-phenyl phenolate*, Sodium-6-chloro-2-phenyl phenolate*, Sodium dodecyl benzene sulfonate*, Sodium ortho-phenyl phenolate*, Sodium saccharin, Sodium silicate, Sodium stearate, Sodium sulfate, Sorbitol, Spermaceti* **, Sulfated Castor oil 75%* **, Tegacid* **, Tricalcium phosphate, Veegum* **, Whitening agents* **, Zinc stearate.

Of the substances for which data were available, none were found to be toxic at the concentrations expected in shower and laundry wastewaters. For these compounds for which data were not available, we must rely on the results of previous Army-sponsored studies on wastewater concentrates.

If it is desired to obtain further data on individual compounds, consideration could be given to:

Obtaining data to fill in the data gaps noted above. The following tests would suffice: single oral LD₅₀ to rats, single skin penetration in rabbits, skin irritancy test, eye irritancy test, and human skin patch test.

Compilation of a master data file containing physical, chemical, engineering treatability, and toxicity data for each substance expected to be present in waters to be treated and reused. Data on sources and expected concentrations could be included. This file could be manual or computer-based. It could be updated as new products are introduced.

ARMY-SPONSORED RESEARCH

Objective

Four reports of Army-sponsored research were evaluated to determine the extent to which they provided information applicable to an assessment of the health effects of shower and laundry water reuse by field Army units. These reports are considered below in chronological order.

Results

R.G. Tardiff and J. L. Mullaney prepared a report on "The Compounds in the 'MUST' Wastewater as They Relate to Hazards in the Product Water: A Toxicity Evaluation", (1973). Industrial product identities and their concentrations were furnished by the U.S. Army Medical Research and Development Command. Tardiff and Mullaney determined the concentrations of product components, performed a literature search and provided lists of: compounds requiring no further study, compounds requiring control (treatment) in product (recycle) water, and compounds requiring additional study. The report was extremely concise but adequately documented with references and appendices. There was no discussion of the basis for compound classification, no discussion of the effects of compound concentration or exposure time on toxic responses, no discussion of the applicability of retrieved data to water reuse, and no discussion regarding the specification of a toxicology protocol for water reuse evaluation. Such matters were left to the judgement of the sponsor. Much of the data presented in the appendices applies to the compounds in concentrated form, and is thus not applicable to evaluations of water reuse.

Of the 104 compounds studied, three which appear in the present report were found to require control (treatment): boric acid, hexachlorophene, and tripolyphosphates. Three compounds were found to require further study: NN diethyl-m-toluamide, isopropyl myristate, and oleic acid.

Bernard P. McNamara prepared a report of "Toxicology Program on Certain MUST Wastewater Components," (1974). Included in his tests of nine compounds (or mixtures) were tests of two substances found in shower wastewaters: pyrogallol and isopropyl myristate. The tests of dermal absorption, skin irritancy, eye irritancy, and skin sensitization were carefully performed according to accepted protocols. The first three tests were performed with pure compounds according to protocols not necessarily applicable to the evaluation of health effects of water reuse. Isopropyl myristate was shown to be not toxic by dermal sorption, not a skin irritant, not an eye irritant, and not a skin-sensitizing agent. At the low concentrations expected in shower wastewaters, it would be expected to be non-toxic by these tests. Pyrogallol was shown to be not toxic by dermal sorption, not a primary skin irritant, a positive eye irritant, and a possible skin sensitizer. At the low concentrations present in shower wastewaters, pyrogallol might or might not be an eye irritant.

Robert B. Grieves and Dibakar Bhattacharyya prepared a report on "Membrane Ultrafiltration to Treat Non-Sanitary Military Wastes," (1976). Their tests of synthetic and actual shower and laundry wastewaters demonstrated that ultrafiltration can be used to effectively treat these wastewaters to provide recyclable water up to at least 90% water conversion. They provided wastewaters, ultrafiltrate (permeate) of treated wastewaters, and concentrates of treated wastewaters to Witherup and Emmett for toxicological investigations.

Sylvan Witherup and Edward A. Emmett prepared a report on "The Toxicity and Irritancy of Ultrafiltrates of Non-Sanitary Military Wastes," (1977). Witherup and Emmett tested the samples provided by Grieves and Bhattacharyya as well as freeze-dry concentrates of the samples. Except for the freeze-dry concentrates, no toxic effects were noted in ocular, dermal, or oral toxicity tests. The freeze-dry concentrates with solids contents in the range of 2 to 9% did produce toxic responses, as might be expected of many substances. Though strictly applicable only to the wastewaters tested, the results of Witherup and Emmett tend to confirm that at least for some wastewaters, neither the untreated nor treated wastewaters produce toxic responses at concentrations expected in field Army situation.

Conclusions on Army-Sponsored Research

Tardiff and Mullaney performed a literature survey involving single examples of each of several types of cleaning and health care products. In the present study, several examples of each of several types of cleaning and health care products were evaluated. In addition, the present study considered the effects of compound concentration and time of exposure on toxic response. For compounds for which data were available, data indicate that no toxic effects are anticipated for oral, dermal or ocular exposures.

McNamara's report provides data on two specific compounds, often for concentrations and exposure times much greater than those expected in recycled waters.

The Grieves and Bhattacharyya study together with the Witherup and Emmett study, provide valuable information on wastewaters and wastewater concentrates. Their results indicate that wastewater reuse is unlikely to result in oral, dermal or ocular toxicity. Though the wastewaters tested were not chemically analyzed and we must infer their compositions, we can be relatively certain that no wastewater constituents presented a serious risk of oral, dermal or ocular toxicity.

OTHER FACTORS AFFECTING REUSE

The present assessment of health effects has led necessarily to a consideration of several other factors affecting the reuse of shower and laundry waters by field Army units. These factors include:

- the different risks and benefits for Army as opposed to civilian situations;
- the relative risks for short-term (1-7 days) and long-term (greater than 7 days) water reuse;
- the engineering state-of-the-art in transportable wastewater treatment systems compatible with field Army operations; and
- the impact of the esthetics of wastewater reuse as perceived by the Army soldier.

A final decision regarding water reuse is the responsibility of the Department of Defense and is beyond the scope of the present study. We may note however, that in a combat situation the risks of water reuse for showers and laundries may be more than offset by the increased availability of ammunition, fuel, and supplies. For the Army soldier, water reuse may result in a significantly increased survival rate under combat conditions. To the extent that water reuse could result in a positive benefit to the Army soldier, it should be considered for field use.

Long-term reuse of water by field Army units for showers and laundries would require the development of both acute and chronic toxicology data. Short-term reuse of water reduces the data requirements to data on acute effects.

The engineering state-of-the-art dictates that for field Army operations, a wastewater treatment system should be transportable, rugged, simple to operate, simple to monitor and control, and not require hazardous or scarce supplies for its operation. In addition, system reliability and water availability must be balanced against product water purity and system cost. These factors will come into play when a choice is made between, for instance, either a UF-RO or pre-treated-ACA-IX system on the one hand and a UF system on the other hand. The former systems would tend to be more complex but with a very high-quality product water. The latter (UF) system could be designed as a compact, simple system (with a discardable UF cartridge), producing water of an adequate but not extremely high quality.

Ethetics may very well be the deciding factor in water reuse. Toxic hazard potential appears to be minimal, especially for short-term reuse. The odor, taste (accidental ingestion), or visual appearance of treated and recycled water will influence the Army soldier's willingness to consider water reuse as a positive benefit. To the extent that this is the case, esthetic criteria as well as chemical criteria may be used to assess the engineering performance of candidate wastewater treatment systems.

SUMMARY AND CONCLUSIONS

Will the Army soldier benefit by water reuse?

In attempting to assess health effects of water reuse, we have learned that at least under certain conditions, limited water reuse is likely to benefit the soldier. Benefits of water reuse can be maximized if, for a carefully defined reuse scenario, we delineate the basis for health effects criteria. What kinds of tests under which conditions, and on which test groups, need to be performed to assess the health effects of water reuse?

Of all possible kinds of water reuse, consideration has been limited to short-term shower and laundry water reuse by field Army units. These are high-volume uses under field Army conditions which can be readily segregated from other uses. Both the wastewaters and the users of the recycled water are well-defined and subject to a degree of control which will minimize health impacts. One need worry neither about septic wastes nor about unusually sensitive water users (e.g., infants, elderly persons, or medical patients). Concern may be focused on possible acute effects, since exposure would be too short to produce chronic effects.

Results achieved to date indicate that it should be possible to treat and reuse shower and laundry waters without any significant health effects. Literature studies have uncovered no data to indicate adverse health effects. Animal and human (skin patch) tests of wastewater and wastewater concentrates indicate no adverse health effects (oral, dermal or ocular) at concentrations up to several times those expected in treated, recycled water.

Since the present evaluation suggests there will be no adverse effects, what further tests are required to establish the safety, without question, of shower and laundry water reuse by field Army units? If the answer is to be obtained under conditions other than battlefield conditions, human clinical trials must be performed.

U.S. Army Medical Research and Development Command Regulation No. 70-25, "Use of Human Subjects in Research Development, Test, and Evaluation," establishes procedures for conduct of human clinical trials. This regulation specifies three conditions which must be satisfied before and during the conduct of human clinical trials: adequate scientific justification, adoption of appropriate measures to minimize risk, and adoption of administrative review procedures.

A human clinical trial is scientifically justified if the benefits of the study (importance of the knowledge to be obtained) outweigh the risks to test subjects, and if the results are not obtainable except by means of a human clinical trial. This is phrased in various ways in paragraphs 1-5-2a, 1-5-2e, and 2-4-2a of the regulation.

Measures to minimize risk to human test subjects include provisions for: qualified personnel to conduct the study (paragraph 1-5-2g and 2-3a); consent of human test subjects (1-5-1); adequate facilities for conducting the test and for administration of any medical treatment that might be necessary (1-5-2f, 2-3b, 2-3d); sufficient animal or laboratory experiments or other evaluations to give assurance of acceptable risk prior to use of human subjects (1-5-2d); a written research proposal to be reviewed either by HQDA (SGRD-RP) in the case of civilian contractors or by HQDA (DASG-RDZ-H) in the case of USAMRDC laboratories (1-12); and research protocols (2-5, 3-4a, 3-8). Of these measures to minimize risk to human test subjects, conditions pertaining to subject consent are covered in the most detail. Consent must be voluntarily given and the subject must be fully informed of the value of the tests and the risks associated with the tests. The subject must be free to withdraw his consent at any time prior to or during the study without penalty. "It should be emphasized that the essence of voluntary, informed consent is a full discussion of the nature of the study by a scientifically competent person with the prospective human subject, in the presence of a witness not directly involved in the project." Thus, the scientific justification for the study and the risks of the study are assessed by: the planners of the study, the qualified persons conducting the study, and the subjects of the study.

Administrative review procedures have been specified to insure that the spirit and letter of the regulations are followed. These procedures include: Surgeon General approval of all studies before they may commence (paragraph 1-6), filing of assurances and certificates prior to study commencement (3-4, 3-5); establishment of a human use committee with continuing review responsibilities (2-4-1, 2-4-2, Appendix D); and detailed procedures for record-keeping to verify compliance with regulations (1-13).

It is recommended that consideration be given to:

(1) Preparation of an assessment of the benefits of short-term shower and laundry water reuse by field Army units. This assessment should be phrased in terms meaningful to members of the U. S. Army Medical Research and Development Command and other Army officials; to members of a human use Committee as defined in USAMRDC Regulation 70-25, paragraph 2-4-1; and to prospective human test subjects for a clinical trial. At least preliminary estimates of acceptable engineering configurations and performance characteristics of suitable wastewater treatment systems should be specified, including a target value for the extent of water recycle.

(2) Preparation of an assessment of acceptable risks associated with short-term shower and laundry water reuse in a combat situation. Topics covered might include digestive tract irritation due to accidental ingestion, skin irritation, and eye irritation.

(3) Preparation of criteria defining acceptable risks for human test subjects in clinical trials of water reuse. Acceptable (and unacceptable) toxic symptoms should be defined together with the minimum professional qualifications to be possessed by the person or persons responsible for evaluation of toxic responses in human test subjects.

(4) Preparation of a protocol for animal studies designed to assess the possibility of human toxic response to shower and laundry water reuse. The protocol must specify the composition of products to be used during showering or laundering, the wastewater treatment system to be used and its mode of operation, chemical analysis to be performed, and responses to be monitored. It should also specify criteria for test adequacy, i.e., criteria for cessation of animal tests and for consideration of human clinical trials.

(5) Preparation of a protocol for human clinical trials of short-term shower and laundry water reuse. It is assumed that human clinical trials would be proposed only after assessments have been completed for: benefits to be expected, acceptable risks in a combat situation, acceptable risks for human test subjects, and likely risks for test subjects as projected from animal studies. The human use protocol should then focus on early detection and treatment of toxic symptoms which might arise during the course of the clinical trials.

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TABLE 1
SHOWER WASTEWATER PRODUCTS

Product	Concentration mg/l
Shower Cleaner	100-220
Salt	60-180
Soap, Deodorant	50-150
Hair Oil	25-150
Soil (Kaolinite)	20-50
Talc	20-50
Hair Shampoo	10-50
DEET	1-20
Epithelium	18
Lactic Acid	5
Urea	1-3
Toothpaste	2
Hair	2
Potassium	1.5
Shaving Preps	1
Disinfectant	1
Lotions	1
Mouthwash	1
Deodorant	1
Suntan Preps	1

TABLE 2
LAUNDRY WASTEWATER PRODUCTS

Product	Concentration mg/l
Sodium Carbonate	499
Detergent Type I	433
Detergent Type II	172
Vegetable Oil	166
Kaolinite Clay	133
Sour (Downey)	116
Urea	13
DEET	12

TABLE 3
LAUNDRY WASTEWATER CONSTITUENTS

	Concentration mg/l
Sodium carbonate	530
Vegetable oil	170
Kaolinite	130
Sodium alkylbenzenesulfonate	120
Sodium sulfate	110
Sodium tripolyphosphate	90
Sodium silicate	80
Sodium fluosilicate	80
Ethoxylated alcohol	60
Urea	13
N, N-Diethyl-m-toluamide	9
Sodium carboxymethylcellulose	6
Protease	3
Fluorescent whitening agents	3
Ethanol	3

TABLE 4
SHOWER WASTEWATER CONSTITUENTS

	mg/l	The following compounds present at <0.2 mg/l
Silica Flour	100 - 210	
Salt	60 - 180	
Castor Oil	20 - 130	Alumina
Isopropyl alcohol	18 - 105	Aluminum chloride
Ethanol	15 - 85	Aluminum sulfate
Kaolinite	20 - 50	Ammonium alum
Oleic acid	16 - 50	Beeswax
Water	10 - 45	Boric acid
Talc	41	Cetyl alcohol
Tallow	13 - 38	Corn starch
Stearic acid	11 - 31	Bentonite
Coconut oil	9 - 30	Hexachlorophene
Castor oil, sulfonated (75%)	6 - 30	Isopropyl myristate
Ultrawet 60-L	5 - 25	Jamaican rum
Ammonium lauryl sulfate	5 - 25	Magnesium carbonate
Sodium lauryl sulfate	5 - 22	Magnesium oxide
Epithelium cells	18	Glycerol monostearate
N,N-Diethyl-m-tolamide	1 - 15	Methyl paraben
Sodium dodecylbenzenesulfonate	3 - 13	Lanolin
Sodium tripolyphosphate	5 - 11	Petrolatum
Olive oil, sulfonated (75%)	2 - 10	PABA
Tannic acid	1 - 8	Isopropyl palmitate
Triethanolamide alkylbenzene sulfonate (60%)	1 - 7	Polyethylene sorbitan mono- stearate
Potassium oleate (20%)	1 - 6	Saccharin sodium
Kaloin, colloidal	5	Sodium-6-chloro-2-phenyl-phen- olate
Lactic acid	5	Sodium hydroxide
Triethanolamine	1 - 5	Sorbo
Urea	1 - 3	Spermaceti
Glycerol	1 - 3	Sorbitan monostearate
Potassium hydroxide	0.7 - 3	Stannous fluoride
Zinc stearate	3	Veegum
Coconut diethanolamine (92%)	0.5 - 3	Zinc chloride
Hair	2	Sodium stearate
Mineral oil	0.5 - 2	
Potassium	1.5	
Calcium carbonate	0.9	
Aluminum hydroxide	0.9	
Sorbitol	0.7	
Dicalcium phosphate	0.6	
Sodium-ortho-phenylphenolate	0.6	
Sodium-4-chloro-2-phenylphenolate	0.5	
Sodium metaphosphate	0.4	
Aluminum formate solution	0.4	
Propylene glycol	0.3	
Tricalcium phosphate	0.2	
Volatile silicone	0.2	
Tegacid	0.2	
Aluminum chlorhydrate	0.2	
Tween 80	0.2	

TABLE 5
CONSTITUENTS OF MIXED WASTEWATER

	mg/l		mg/l
Sodium carbonate	240	Mineral oil	0.2-0.9
Silica flour	50-110	Potassium	0.8
Salt	33-100	Calcium carbonate	0.5
Kaolinite	70-90	Aluminum hydroxide	0.5
Vegetable oil	75	Sorbitol	0.4
Castor oil	12-70	Dicalcium phosphate	0.3
Isopropyl alcohol	10-60	Sodium ortho-phenyl-phenolate	0.3
Sodium alkyl-benzenesulfonate	55	Sodium 4-chloro-2-phenylphenolate	0.3
Sodium sulfate	51	Sodium meta-phosphate	0.2
Ethanol	10-50		
Sodium tripolyphosphate	40-45	Compounds present at <0.2 mg/l	
Sodium silicate	35	Alumina	
Sodium fluosilicate	35	Aluminum chlorhydrate	
Ethoxylated alcohol	30	Aluminum chloride	
Oleic acid	10-30	Aluminum formate	
Water	5-25	Aluminum sulfate	
Talc	20	Ammonium alum	
Tallow	1-21	Beeswax	
Stearic acid	6-17	Boric acid	
Coconut oil	5-16	Cetyl alcohol	
Castor oil, sulfonated	3-16	Corn starch	
Ultrawet 60L	3-14	Bentonite	
Ammonium lauryl sulfate	3-14	Hexachlorophene	
N,N-Diethyl-m-toluidine	5-12	Isopropyl myristate	
Sodium lauryl sulfate	2-12	Isopropyl palmitate	
Epithelium cells	10	Jamaican rum	
Urea	6-8	Magnesium carbonate	
Sodium dodecyl benzenesulfonate	1-7	Magnesium oxide	
Olive oil, sulfonated	1-5	Glycerol monostearate	
Tannic acid	0.7-5	Methyl paraben	
Triethanolamide alkyl-benzenesulfonate	0.7-4	Lanolin	
Potassium oleate	0.7-3	Petrolatum	
Kaolin, colloidal	3	PABA	
Sodium carboxymethyl-cellulose	3	Propylene glycol	
Lactic acid	3	Polyethylene sorbitan monostearate	
Triethanolamine	0.5-2	Saccharin sodium	
Glycerol	0.8-2	Sodium-6-chloro-2-phenylphenolate	
Potassium hydroxide	0.4-1	Sodium hydroxide	
Zinc Stearate	1	Sodium stearate	
Coconut diethanol-amine	0.3-1	Spermaceti	
Fluorescent whitening agents	1	Sorbitan monostearate	
Hair	1	Stannous fluoride	
		Tegacid	
		Tricalcium phosphate	
		Tween 80	
		Veegum	
		Volatile silicone	
		Zinc chloride	

TABLE 6. CONCENTRATIONS OF CHALLENGE/PROBLEM CONTAMINANTS
IN TREATED WASTEWATER PRIOR TO REUSE FOR AN
INFINITE NUMBER OF RECYCLES (80% CONVERSION)

Contaminant	X ₁ (mg/liter)	x _∞ y (mg/liter)			
		UF-RO	UF	Pretreat ACA/IX	ERDLator
<u>Shower</u>					
Ethanol	84	82	180	160	240
Isopropanol	105	9	89	59	130
Glycerol	3	0	3	2	5
Lactic Acid	5	2	6	2	4
Triethanolamine	8	0	6	2	5
Urea	3	2	6	6	10
Sodium Chloride	180	2	130	2	720
<u>Laundry</u>					
Ethanol	1	1	3	3	4
Urea	13	11	25	28	44
Sodium Carbonate	533	0	360	0	2,100
Sodium Sulfate	112	1	78	1	450
<u>Composite</u>					
Ethanol	47	46	100	89	130
Isopropanol	57	5	49	32	73
Glycerol	2	0	1	1	3
Lactic Acid	3	1	3	1	2
Triethanolamine	5	0	4	1	3
Urea	8	6	15	16	25
Sodium Chloride	98	1	68	1	390
Sodium Carbonate	242	0	160	0	960
Sodium Sulfate	51	1	35	108	200

TRACE ELEMENT ANALYSES OF SEVERAL ADVANCED
WASTEWATER TREATMENT PLANT EFFLUENTS

Herbert R. Pahren
Physical Science Administrator
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

Nancy S. Ulmer
Research Chemist
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

Advanced wastewater treatment (AWT) processes will undoubtedly be used in any system designed to produce potable water from wastewater. Generally, laboratory analyses to determine the concentration of the elements in AWT processes are limited to a few common metals such as those listed in the National Interim Primary Drinking Water Regulations.¹ The dozens of other elements are assumed not to be present in concentrations at harmful levels. Many of these elements would be very difficult to analyze.

A new multielement procedure, involving proton-induced X-ray emission (PIXE) technology,² was utilized for the simultaneous determination of 70 elements heavier than aluminum in the effluent of four AWT full scale plants or pilot plants. Plants cooperating in this short term screening study were: Orange County, California, Water Factory 21; South Lake Tahoe, California; Pomona AWT Pilot Research Facility, Pomona, California; and the US EPA pilot plant at Blue Plains, Washington, D.C. The objective of this screening was to determine if any element which is not ordinarily checked might be present in the effluent at a level which would be of a health concern if the water were ever reused for potable purposes. At the Pomona plant, influent samples were obtained as well as the effluent so that removals through the system could be determined.

METHODS

One-quart grab samples were taken and preserved with 1.5 ml concentrated HNO₃/1. The polyethylene cubitainers containing the samples were mailed to the US EPA laboratory in Cincinnati. Three samples were taken at each location over a five-month period during the last half of 1975. Periodically, a number of samples were shipped to the US EPA contractor, Purdue Research Foundation, in West Lafayette, Indiana where the analyses were performed.

The multielemental analysis by PIXE consisted of (1) preparation of targets from a 1-ml (for calcium and potassium only) and a 30-ml sample aliquot using a unique vapor filtration technique, (2) bombardment of each target using a pulsed beam of 4-MeV protons from a FV tandem Van de Graaf accelerator, (3) detection of emitted X-rays by a Kevex model 3010 silicon-lithium detector, (4) transmission of the electrical pulses from the detector to a PDP 15/40 computer, (5) conversion of the pulses to a number proportional to the energy of the X-ray, and (6) calculation and recording of the concentration of the elements by the computer

system which had been programmed to compensate for various interferences, know the precise energy of all X-rays of interest and the shapes of the peaks, check and adjust the energy calibration, perform a multidimensional linear least-squares fit to the data, check on the relative intensities of the X-rays from any given element and record any discrepancies along with the elemental results. In general, PIXE is useful for detection of elements heavier than aluminum, but results to date indicate that other methods are better for measuring bromide, chloride, iodide, phosphorus, and sulfur. Details of the PIXE procedure are contained in the contractor's report.²

RESULTS AND DISCUSSION

Elements with at least one positive result among the fourteen effluent samples from the four AWT plants are shown in Table 1. Elements not detected are presented in Table 2. It should be mentioned that for samples with greater than 50 ppm calcium, as was the case with each of the AWT samples, the cited detection limit is multiplied by the factor (calcium concentration in ppm)/50. This modification does not apply to scandium.

When comparing the results to the EPA National Interim Primary Drinking Water Regulations (Table 3), it may be seen that these elements are generally well below the limits. Only one sample exceeded the limits. Lead at the Blue Plains pilot plant had a value of 63 ppb, slightly over the limit of 50 ppb for drinking water, at a time when the plant was being shut down. Earlier values of 2.8 and 4.1 ppb were obtained under normal operating conditions. Although the maximum permissible mercury level is 2 ppb, the PIXE detection limit is 3 ppb. Since mercury was not detected in any of the samples, it is probable that mercury was below the limit for drinking water.

At the Pomona pilot plant, samples were taken of the AWT influent as well as the effluent of two parallel treatment systems. One system consisted of coagulation, sedimentation, filtration, chlorination, and carbon adsorption. The other had two carbon adsorption steps with intermediate chlorination. Table 4 lists the removals of those elements found in the wastewater. In certain instances, elements increased during treatment. Both iron and manganese increased significantly in each Pomona treatment system. The unusually high maximum iron value in Table 1 was from the Pomona plant.

In general, results of this screening exercise indicate that trace elements were not a problem at the four advanced wastewater treatment plants. Elements which are not normally checked were either below detectable limits or not of a level to be of serious concern.

REFERENCES

1. National Interim Primary Drinking Water Regulations. EPA-570/9-76-003., Ofce. of Wtr. Sply. Washington, D.C. (1976).
2. Simms, P.C. & Rickey, F.A. The Multielemental Analysis of Drinking Water Using Proton-Induced X-ray Emission (PIXE). EPA-600/1-78-058., U.S. Environ. Prot. Agency. Cincinnati, OH. (Sept. 1978).

TABLE 1

Elements Detected in Advanced Wastewater Treatment Effluents

Element	Detection Limit ppb	Percent Detected	Minimum ppb	Median ppb	Maximum ppb
Potassium	100	100	6,530	8,780	17,600
Calcium	100	100	52,500	64,400	96,000
Vanadium	1	57	<1.0	2.0	7.5
Chromium	0.1	57	<0.1	0.7	12
Manganese	1	71	<1.0	7.7	412
Iron	1	100	18	33.5	17,900
Cobalt	0.1	7	<0.1	<0.1	0.9
Nickel	0.1	100	2.2	9.3	47
Copper	1	93	<1.0	7.1	64
Zinc	1	100	12	26	3,620
Molybdenum	1	79	<1.0	4.1	15
Silver	1	7	<1.0	<1.0	2.4
Cadmium	0.1	36	<0.1	<0.1	5
Lead	0.1	71	<0.1	2.7	63
Titanium	1	36	<1.0	<1.0	33
Gallium	0.1	29	<0.1	<0.1	0.9
Rubidium	0.1	93	<0.1	6.1	11
Strontium	0.1	100	91	432	604
Tin	0.3	21	<0.3	<0.3	7.3
Antimony	0.4	21	<0.4	<0.4	9.8
Barium	2.5	93	<2.5	35.5	77
Thallium	0.3	7	<0.3	<0.3	0.5
Arsenic	0.5	93	<0.5	2.3	9.5
Selenium	1	29	<1.0	<1.0	2.5

TABLE 2

Elements not Detected in Advanced Wastewater Treatment Effluents

Element	Detection Limit ppb	Element	Detection Limit ppb
Scandium	0.1% Ca	Germanium	0.1
Silicon	5000	Palladium	0.1
Indium	0.2	Tellurium	0.7
Cesium	2	Lanthanum	2.5
Tungsten	0.3	Platinum	0.3
Niobium	0.1	Bismuth	0.3
Ruthenium	1	Zirconium	1
Rhodium	1	Cerium	20
Praseodymium	18	Neodymium	16
Promethium	13	Samarium	12
Europium	11	Gadolinium	9
Terbium	8	Dysprosium	7
Holmium	6	Erbium	5
Thulium	3	Ytterbium	3
Lutecium	3	Hafnium	3
Tantalum	3	Rhenium	3
Osmium	3	Iridium	3
Gold	3	Mercury	3
Polonium	3	Astatine	3
Francium	3	Radium	3
Actinium	3	Thorium	3
Protactinium	3	Uranium	3
Neptunium	3	Plutonium	3

TABLE 3

Comparison of Advanced Wastewater Treatment Effluents
With Drinking Water Standards
(All Values in ppb)

Element	Minimum	Median	Maximum	NIPDWR*
Arsenic	<0.5	2.3	9.5	50
Barium	<2.5	35	77	1,000
Cadmium	<0.1	<0.1	5.0	10
Chromium	<0.1	0.7	12	50
Lead	<0.1	2.7	63	50
Mercury	<3	<3	<3	2
Selenium	<1	<1	2.5	10
Silver	<1	<1	2.4	50

*National Interim Primary Drinking Water Regulations

TABLE 4
Percent Removal of Elements at Pomona

Element	Carbon System	Chemical Coagulation System
Potassium	8	3
Calcium	0	0
Vanadium	0	0
Chromium	78	99
Manganese	0	0
Iron	0	0
Nickel	42	44
Copper	90	88
Zinc	42	21
Molybdenum	33	63
Lead	92	99
Titanium	67	67
Rubidium	29	52
Strontium	3	2
Tin	67	100
Barium	20	18
Arsenic	21	0

A LABORATORY STUDY OF VIRUS UPTAKE BY MINERALS AND SOILS

G. Wolfgang Fuhs
Director, Environmental Health Center

Rebecca S. Moore
Research Scientist I

Michael M. Reddy
Research Scientist IV

Lawrence S. Sturman
Research Physician

Dene H. Taylor
Postdoctoral Fellow

Division of Laboratories and Research
New York State Department of Health
Empire State Plaza
Albany, New York 12201

INTRODUCTION

When wastewater effluents are used for groundwater recharge and wastewater sludges are applied to land, the public health must not be compromised. Particularly, the risk of human exposure to pathogenic viruses must remain low, especially in regard to other modes of viral transmission. This requires that the acceptability of disposal sites and conditions be determined in advance for each proposed project. To develop the required predictive capacity it is necessary that the fundamental mechanisms of virus interactions with soils are understood for a wide spectrum of both viruses and soils.

This paper is a preliminary report of a laboratory study of the interactions of radiolabelled poliovirus (type 2) with over 40 representative soils and minerals. Substrates include members of 6 of the 10 accepted soil orders with a wide variation of organic content, pH, surface area and chemical reactivity. Further intensive studies are being carried out with poliovirus and a selected group of substrates under differing conditions. Similar studies with 4 other virus strains of public health significance are in progress. A full report of the poliovirus studies is in preparation for presentation to the U.S. Environmental Protection Agency.

MATERIALS AND METHODS

Over 30 different dry soils, minerals and rocks were obtained from Ward's Natural Scientific Establishment, Rochester, NY. When necessary these were ground with either a steel disc grinder or a tungsten carbide ring mill and the fraction passing a 0.15 mm sieve was taken. In addition, a number of other soils, mostly from New York

State, were obtained; these were air-dried and that part which passed a 2 mm sieve was collected. The substrates are being characterized by elemental analysis, selective chemical extraction, interaction with dilute electrolyte solutions, measurement of polyelectrolyte adsorption capacities and some physical properties. Ottawa sand, 0.6 to 0.8 mm, meeting ASTM C190 specifications was chosen as the reference material for all studies.

Poliovirus type 2 is produced and grown in monolayer cultures of HeLa cells in the presence of ^3H or ^{14}C labelled amino acid mixtures. The virus is concentrated by centrifugation and purified by isopycnic sedimentation in a CsCl gradient followed by dialysis against phosphate-buffered saline. Infectivity is determined by plaque titration on HeLa cell monolayers. Radioactivity is measured by liquid scintillation spectrometry. Full details of these procedures will be made available elsewhere.

A synthetic electrolyte solution resembling secondary-treated municipal wastewater effluents of New York State was chosen as the suspending liquid. The concentrations of the various salts in this medium are CaCl_2 , 1.0 mM; MgSO_4 , 0.375 mM; KCl , 0.1 mM; and NaHCO_3 , 1.25 mM. Immediately prior to experimentation, the pH of this solution is adjusted to 7.0 with dilute HCl .

The interaction of virus and substrate is carried out in a batch process. An aliquot of suspending medium is added to the substrate (0.5 g) in a polypropylene centrifuge bottle. A further aliquot of the medium containing between 10^6 and 10^9 pfu of poliovirus is added to give a final volume of 2.0 ml. The amount of radioactivity added is usually between 10^3 and 10^6 cpm. The bottles are closed and shaken at 300 rpm on a gyratory shaker at 4°C for one hour. The unadsorbed virus is separated from the substrate by centrifugation at 10,000 rpm in a Sorvall GSA rotor for 15 min at 4°C . The pH of the supernatant is measured and aliquots taken for virus titration and measurement of radioactivity. On each occasion control samples (without substrate) are prepared, and the interaction with the reference material, unground Ottawa sand, is also determined. The amount of material adsorbed is taken as the difference between the amount added, or that determined for suitable control experiments, and that remaining in suspension.

RESULTS AND DISCUSSION

The results so far obtained from this work have shown that the adsorption of poliovirus to various materials is rapid and has reached a steady-state within one hour. Any further changes after that time are apparently caused by slow reactions of the suspending medium as the substrate produces variations in the solution properties, notably changes in the pH. Studies with Ottawa sand have shown that the adsorption at pH 7.4 to 7.6 may be described by the Freundlich equation at low virus concentrations and by the Langmuir equation when surface saturation is being approached. The surface concentration at saturation is between 2 and 3×10^{11} pfu/m ^{2} which is approaching mono-

layer coverage. Further studies have shown that pH is extremely important to the adsorption process - binding is not favored in alkaline media but it is in neutral and acid systems. In addition, the presence of Ca^{2+} greatly enhances adsorption to unreactive surfaces, such as silica sand, from neutral media. When bound, poliovirus retains much of its viability and some bound virus may be desorbed by dilution in the adsorbing medium. That is, adsorption is at least partially reversible with little change in viral specific infectivity. Distilled water was a more effective eluent than the previously described synthetic freshwater which again emphasizes the importance of simple electrolytes in virus adsorption. In all studies with Ottawa sand the determinations of the concentrations of viral material by radioactivity and infectivity measurements have been similar, indicating that inactivation, degradation and aggregation have been unimportant.

Of the wide range of other materials studied only a few have shown adsorption of virus of less than 95% or more than 99.9% based on infectivity changes. Preliminary studies indicate that there is little correlation between uptake of virus and most substrate properties for all substrates taken as a group. However, strongly basic materials appear to be poor adsorbents, as do materials with moderate to high contents of organic matter. Sands overgrown with bacteria, and hence coated with bacterial exopolymer, were compared with untreated sands. In each case the coated sands have been markedly weaker adsorbents. When the high surface concentration needed for saturation with virus is considered it would appear unlikely that any mineral surface in contact with a sewage effluent would become saturated with virus. It is more likely that the inability of a soil to retain a virus like poliovirus from an effluent will result from kinetic factors due to rapid passage through the soil into the groundwater, to unfavorable solution electrolyte concentrations or to surface saturation with competing organic matter either present naturally or deposited from the effluent.

The purpose of using radiolabelled virus was to allow the fate of viral material, as well as viral infectivity, to be followed. In this manner virus inactivation, degradation or aggregation might be distinguished from the physical removal of viral material. Significant differences between radioactivity and infectivity determination of the concentrations of viral material have occurred consistently in only a few cases. The causes of these differences are under investigation.

Acknowledgment

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TABLE 1
Brief Characterization of the Soils and Soil Components

Soils

Muck. Dominated by organic matter (ca 30%). Proto-peat.
Genesee Silt Loam. A surface soil containing a moderate amount of organic carbon.
Colonie Sandy Loam. A surface soil containing a relatively high organic content.
Riverhead Sandy Loam. (subhorizons B₃₁) Brookhaven National Laboratories in Long Island. This is a acid outwash soil typical of Long Island, NY.
Genesee Silt. Typical soil material from Monroe County, NY, but contains only a little organic matter. Some carbonate.
Loess. This material is low in carbon and hence in organic matter and carbonates.
Adobe. Contains about 1.2% organic carbon.

Sands

Ottawa Sand. The reference material (20-30 mesh) met ASTM C190 specifications (VWR Scientific, Rochester, NY, no. 20117).
Beach Sand. A calcareous sand with organic matter present.
Dune Sand (Aurain, Mich.). A siliceous sand.
Dune Sand (Monroe, NY). A calcareous sand with some organic matter.
Magnetite Sand. About 80% of the sand is oxide; magnetite, hematite and ilmenite.

Sedimentary Materials

Kame Conglomerate. A calcareous material.
Arenaceous Shale. Moderate carbonate content in addition to the clay mineral components.
Bituminous Shale. Contains both organic and inorganic carbon.
Fossiliferous Limey Shale. Calcium carbonate content of this material exceeds 5%.
Brown Sandstone. A typical sandstone containing 2% calcium carbonate.
Calcareous Shale. A shale enriched in calcium carbonate.
Argillaceous Shale. Predominantly aluminosilicate but contains some calcium carbonate.
Illitic Shale. A calcareous material with a significant clay mineral content.

Minerals

Quartz. Relatively pure (although it was contaminated with iron particles during the preparation.)
Dolomite. Mostly $\text{CaMg}(\text{CO}_3)_2$, but about 10% is not acid soluble; low organic content.
Quartz Conglomerate. Mostly SiO_2 , but contains other matter including a small amount of carbonate.
Calcite. Predominantly CaCO_3 , it contains a significant amount of other materials. It has very little organic matter.

Clays

Dickite. This contains only a small amount of carbon which may be mostly inorganic.
Kaolin. About 93% kaolinite, only a trace of carbon.
Kaolinite. A relatively pure sample exhibiting a low pH in natural water.
Montmorillonite (Wy). An aluminosilicate, with some hy- rous oxides, low in organic carbon, and has a low IEP.
Montmorillonite (Ar.). An aluminosilicate, with some hydrous oxides, low in organic carbon, and has a low IEP.
Halloysite. A relatively pure sample exhibiting a low pH in natural water.
Glaucinite. Appears to be relatively pure.
Attapulgite. This material appears to be predominantly aluminosilicate.

TABLE 2
Poliovirus Uptake by Soils and Soil Components

Substrate	Specific Surface Area m ² /g	Percent of Initial Virus Content Adsorbed*		Substrate Location
		Radioactivity Assay	Infectivity Assay	
<u>Soils</u>				
Muck	1.18	90.0	66.5	Wayne County, New York
Genesee Silt Loam	4.18	95.2	79.1	
Colonie Sandy Loam	5.32	96.1	87.6	Industry, Monroe County New York
Riverhead Sandy Loam	8.92	99.1	98.0	Irondequoit, Monroe County New York
Genesee Silt	10.90	99.1	95.6	Brookhaven National Labs, Long Island, New York
Loess	25.10	99.6	97.5	Industry, Monroe County New York
Adobe	33.80	98.9	89.2	Kansas City, Kansas
				Deming, New Mexico
<u>Sands</u>				
Ottawa	0.01	66.7	88.6	Rochester, New York
Beach	0.73	75.2	99.7	
Dune (MI)	1.51	90.2	95.3	Auburn, Michigan
Dune (NY)	4.09	93.4	97.5	Monroe County, New York
Magnetite	5.74	95.0	98.2	Marysville, California
<u>Sedimentary Materials</u>				
Kame Conglomerate	3.06	96.0	99.3	Monroe County, New York
Arenaceous Shale	3.52	89.0	98.9	
Bituminous Shale	3.78	88.0	95.0	Green County, New York
Fossiliferous Limey Shale	4.53	92.0	99.9	LeRoy, New York
Brown Sandstone	5.56	88.5	99.1	Summit, New York
Calcareous Shale	9.39	87.0	96.6	Medena, New York
Argillaceous Shale	28.90	84.0	99.4	Lima, New York
Illite-bearing Shale	55.00	94.0	99.6	Rochester, New York
				Rochester, New York
<u>Minerals</u>				
Quartz	0.54	82.7	92.7	Penfield, New York
Dolomite	0.85	95.4	99.0	
Quartz Conglomerate	3.19	95.8	99.8	Olean, New York
Calcite	4.03	96.3	98.3	Rosiclare, Illinois
<u>Clays</u>				
Dickite	2.73	96.7	99.3	St. George, Utah
Kaolin	12.00	99.4	99.2	
Kaolinite	16.40	99.2	99.8	Macon, Georgia
Montmorillonite (WV)	31.73	98.0	89.0	Bath, South Carolina (Lamar Pit)
Montmorillonite (AZ)	41.50	97.1	98.3	Clay Spur, Wyoming
Hallowsite	42.00	98.4	98.9	Cameron, Arizona
Glaucanite	59.50	81.0	83.0	Eureka, Utah (Dragon Iron Mine)
Attapulgite	119.00	97.6	95.6	Birmingham, New Jersey
				Attapulgus, Georgia

* Each experiment included at least two samples.

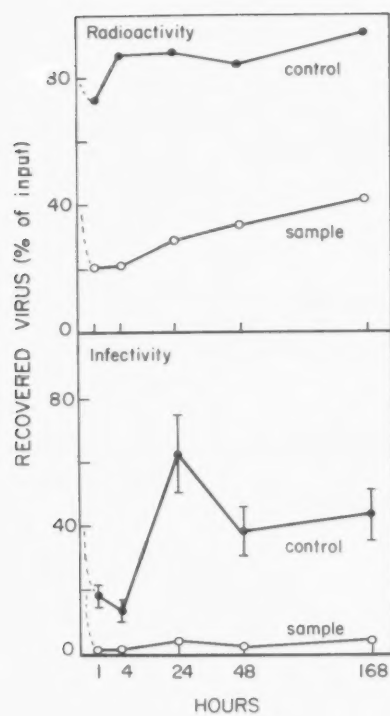


Figure 1. Influence of time on the adsorption of polio-virus by Ottawa sand. At time zero all virus present is assumed to be in the suspended state, i.e., recovered virus = 100%.

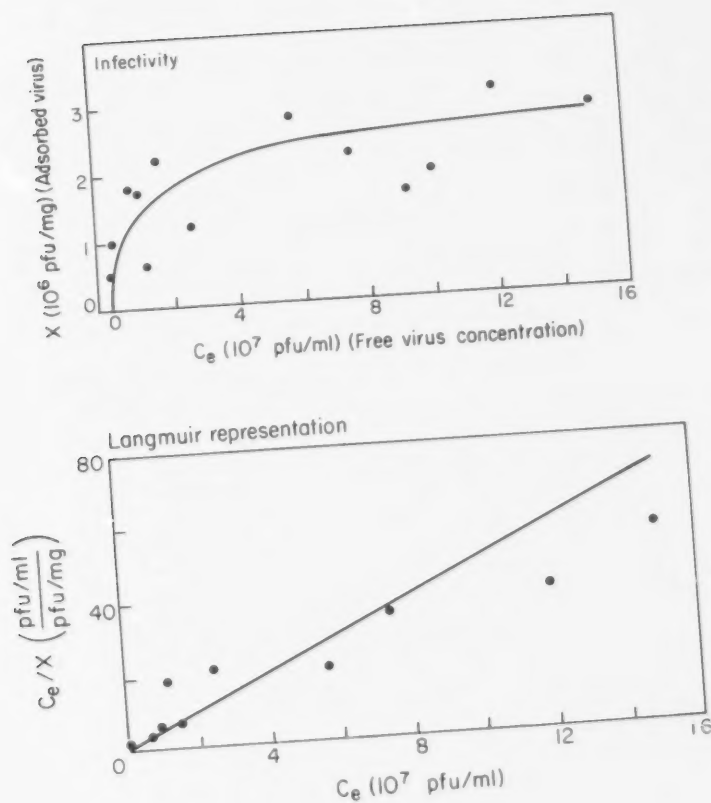


Figure 2. Influence of virus concentration on the adsorption of poliovirus by Ottawa sand near surface saturation. X is the amount of virus adsorbed per unit weight of adsorbent, and C_e is the equilibrium virus concentration in suspension. The least squares regression equation of the Langmuir form corresponds to a maximum surface concentration of about 2.5×10^6 pfu/mg.

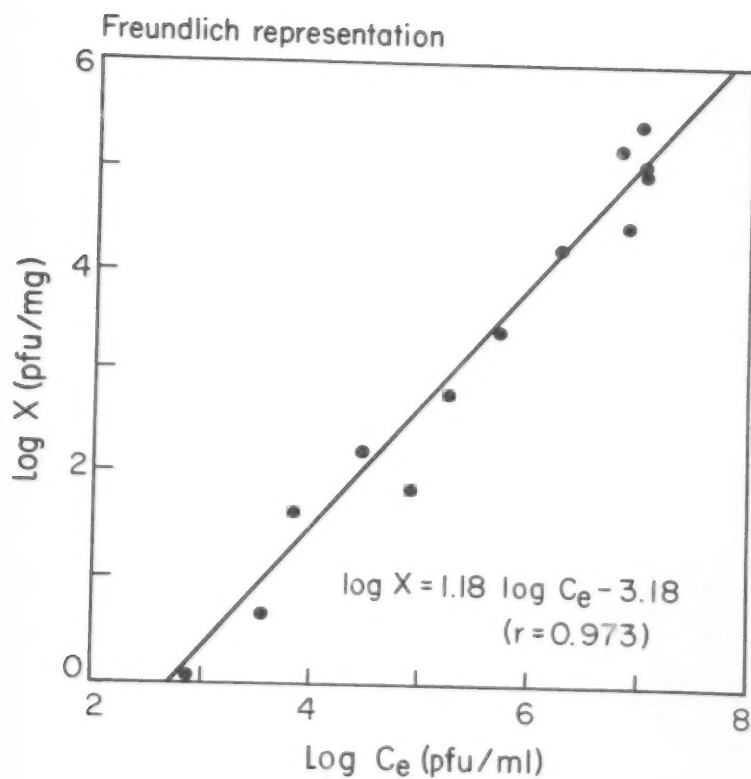


Figure 3. Adsorption of poliovirus by Ottawa sand at low virus concentrations. X is the amount of virus adsorbed per unit of adsorbent, and C_e is the equilibrium virus concentration in suspension.

VIRUS REMOVAL FROM SEWAGE DURING HIGH RATE LAND FILTRATION

J. C. Lance
Soil Scientist

U. S. Water Conservation Laboratory, USDA-SEA-AR
4331 East Broadway, Phoenix, Arizona 85040

C. P. Gerba
Virologist

Department of Virology and Epidemiology
Baylor College of Medicine, Houston, Texas 77030

The increasing demand for water and the increasing volume of sewage water available for renovation combine to make sewage a valuable water resource for the future. Land application is an economical way to renovate sewage water by removing a wide range of contaminants. When large volumes of waste water were applied to relatively small land areas in a high rate groundwater recharge system, chemical and biological reactions effectively removed most of the organic matter, fecal coliforms, nitrogen and phosphate under the proper management systems.¹⁻³ Virus removal from sewage water before reuse is important because certain diseases can be transmitted by water borne viruses and the infective dose is low. Reviews of a number of field and laboratory studies have shown that many soils have a high virus adsorption capacity.⁴⁻⁶ However, only a few field studies have been conducted and the experimental methods used in many laboratory studies make it difficult to extrapolate laboratory data to field conditions. Many soils have not been studied in either laboratory or field experiments.

Previous wastewater renovation experiments with our laboratory soil columns showed that they were good models of the Flushing Meadows groundwater recharge project with respect to removal of fecal coliforms, nutrients and organic material.⁷ In a preliminary experiment Gilbert et. al. did not detect viruses in samples taken every 2 months during 1 year from wells at the Flushing Meadow's project. Then the laboratory columns and batch studies with small soil samples and field experiments were used to study virus movement, adsorption, and survival in various soils. The experiments were designed to study the adsorption pattern and movement of viruses through the soil, to study the effect of many factors such as flow rate, virus concentration, and soil characteristics on virus adsorption and movement and to evaluate the usefulness of poliovirus as a model for adsorption and movement of other enteroviruses.

PROCEDURE

Soil Column Construction

Soil columns that had been intermittently flooded for 4 years during various wastewater renovation experiments were used for the virus movement studies. The columns consisted of 2.75 cm long polyvinyl chloride pipes (10 cm inside diameter) filled with 6 cm of pea gravel and 250 cm of loamy sand (89% sand, 8% silt, and 3% clay) taken from groundwater recharge basins in the dry Salt River bed. Sewage water was applied with a Mariotte siphon to maintain a 15 cm constant pressure head at the soil surface. The flow rate and cumulative outflow through the columns was monitored by weighing the outflow daily. Details of the flow system were described previously.⁷

Another column was packed with loamy sand from the Salt River bed that had not been flooded with sewage water previously. Finally, a column was packed with a coarse sand (93% sand, 6% silt and 1% clay) from the Salt River bed to obtain high flow rates.

Virus Application

Poliovirus type 1 (LSc) was grown, purified and assayed by the plaque forming unit (PFU) method by BGM cells as previously described.⁹ A virus suspension was mixed with dechlorinated secondary sewage effluent to obtain concentrations ranging from 2 to 5×10^4 PFU/ml and the soil columns were flooded (usually on schedules of 9 days flooding alternated with 5 days drying). Samples (2 to 5 ml) were extracted daily from ceramic samplers at depths of 2, 5, 10, 20, 40, 80, 160, and 240 cm and from the outlet line at 250 cm. The experiment was repeated using primary instead of secondary effluent. The adsorption patterns of poliovirus type 1 and echovirus types 1 and 29 were compared in similar experiments using secondary effluent.

Soil columns were flooded with poliovirus seeded sewage water for 2 days, dried for 2 hours and flooded with solution containing 0, 1, and 3mM CaCl_2 . One column was flooded with virus seeded sewage water for two days, then 10 cm of deionized water was infiltrated followed by sewage water with no added virus. In another experiment a column was flooded with virus enriched sewage water for 2 days and then dried for either 2 hours or 1, 3 or 5 days before flooding with deionized water. The column was sampled at the same depths as described above.

Different Poliovirus Concentrations

A soil column was flooded during successive 3 day flooding periods with secondary effluent containing poliovirus concentrations of about 1×10^2 , 3.9×10^3 and 2.6×10^4 PFU/ml. Samples were extracted at various depths during each day of flooding.

Flow Rate Studies

The column packed with coarse sand was flooded with secondary effluent containing about 3×10^4 PFU/ml poliovirus. The height of the column outlet was adjusted to different levels to give flow rates of 0.6, 1.2, 2.4, 4.0, and 12.0 m/day. Samples were extracted at different depths during flooding at each flow rate.

Field Samples

Soil samples were taken from groundwater recharge basins at the Flushing Meadows site that had been flooded with secondary effluent and from another basin in the Salt River bed that had been flooded with primary effluent. Samples were taken on day 1 or 3 after the end of a flooding cycle. The samples were frozen immediately after collection and stored at -70°C until they were eluted about 4 months later. Virus was eluted from the soil by a method recently described by Hurst and Gerba.¹¹

Laboratory Batch Studies

Adsorption of virus to soils was determined by adding 2 ml virus suspensions containing 10^6 to 10^7 PFU to 2 g of soil. The mixture was agitated on a shaker table for 30 min. at 200 rpm. Then the mixture was centrifuged for 5 min. at 3000 rpm and the supernatant was assayed for virus. A control suspension of virus without soil was treated in the

same manner and differences between the control titer and the sample titer were taken as the quantity of virus adsorbed to the soil. The degree of adsorption of 27 reference virus strains by Flushing Meadows recharge basin soil (FM soil) was measured using suspensions of virus in deionized water. The reference strains were poliovirus types 1 to 3; echovirus types 1 to 8, 11 to 13, 22, 24 to 29, and 31; and coxsackie virus types B1 to B6. Several enteroviruses isolated from groundwater beneath a wastewater land disposal site were also studied. These included five strains of echovirus type 1; two strains of a coxsackie virus type B4 and four strains of poliovirus type 3. Since most of the reference viruses adsorbed very well to the FM soil with more than 90% of the added virus adsorbing to the soil, the exceptions were selected for further study with other solutions and soils. These included Echo 1, 12 and 29, Echo 7, Polio 1, and Coxsackie B3 (good adsorbers) were used for comparison. Recent isolates except for poliovirus strains showed more variability and four Echo strains and two Coxsackie strains were selected for further study. The adsorption of the twelve selected virus strains and types to FM soil suspended in deionized water, secondary sewage effluent, soil extract (prepared by shaking 50 g soil in 50 g deionized water and centrifuging) 0.001 M CaCl_2 and 0.01 M CaCl_2 was tested. Then adsorption of nine selected strains and types to nine different soils was tested using suspensions in deionized water. Some characteristics of the nine soils are shown in Table I, and additional information on the soils can be found in a report by Enfield et. al.,¹⁰ describing phosphate adsorption studies with these soils.

RESULTS AND DISCUSSION

Poliovirus Adsorption

Most of the poliovirus was adsorbed in the top 5 cm of the soil columns (Fig. 1). Viruses were detected in only three of 43 1-ml samples from the 150 cm depth and none were detected in 1 ml samples from the 240 and 250 cm depths.⁹ Viruses were detected in five of 43 100ml samples of the daily cumulative drainage from the columns. The virus concentration in the sewage water was reduced by about 2 logs (from a starting concentration of 3×10^6 PFU/ml) during the first 2 cm of travel but an additional 158 cm of travel was required for another 2 log reduction. Flooding a column with the sewage water-virus mixture for 27 days did not saturate the surface layer of soil with virus.

When a column packed with soil which had not been flooded with sewage water previously was flooded with virus seeded sewage, most of the viruses still were adsorbed near the soil surface. This indicated that the concentration of viruses near the surface of the soil columns was not due to a build up of organic material or salts near the soil surface.

Poliovirus Desorption

Flooding columns with deionized water immediately after application of the sewage water virus mixture caused considerable virus desorption and movement. Most of the viruses were readsorbed lower in the column but a few were detected in the column outflow after 2 days of flooding with deionized water. Only a few viruses were detected in profile samples when 1 or 3 mM CaCl_2 solution was applied instead of deionized water and none were detected below the 80 cm depth.

When a soil column was flooded with the virus seeded sewage water followed by 10 cm of deionized water and then by sewage effluent with no

added virus, a peak concentration of 1000 PFU/ml was detected at the 10 cm depth in samples extracted one hour after the sewage alone was applied (or 1 hour after the 10 cm of deionized water infiltrated). No virus were detected in 100 ml samples of the cumulative outflow taken 1 and 2 days after application of the deionized water.

Desorption of virus by deionized water was greatly reduced when the soil was dried for 1 day between the application of virus enriched sewage and deionized water and no viruses were detected when the drying time was increased to 5 days. No viruses were detected below the 80 cm depth when the column was dried at least 1 day between the virus seeded sewage and deionized water applications.

Thus, these experiments show that virus movement due to rainfall would be slight unless rainfall occurred during the first day of a drying period after application of sewage. Heavy rains during the first few hours of the drying period could cause some virus movement but would not move viruses through 250 cm of the Flushing Meadows sand. If rain fell during the first few hours of the drying period, virus movement probably could be almost completely eliminated if sewage application was resumed immediately or if salts were applied to the basins. However desorption may be different for other soils and further work on the subject is needed. Wellings et. al.¹² reported that heavy rains caused virus movement through a soil used for land application of sewage in Florida.

Comparison of Poliovirus Adsorption From Primary and Secondary Effluents

After studying adsorption and desorption of viruses in soil columns some factors which might affect virus adsorption and movement were examined. The effect of different organic matter concentrations on virus adsorption was studied using primary and secondary effluents. Adsorption of poliovirus in both primary and secondary sewage conformed to a Freundlich isotherm when small soil samples (2g) of FM soil were used in batch studies. Difference in virus adsorption from primary and secondary effluent were slight. When a soil column was flooded with virus seeded secondary effluent and later with virus seeded primary effluent, the adsorption patterns were quite similar also. Thus, the organic carbon in the primary effluent (69.5 mg/l for the primary vs 10 mg/l for the secondary) had little effect on virus adsorption.¹³

Adsorption at Different Poliovirus Concentrations

When sewage water was seeded with different virus concentrations the number of viruses found at each depth in the soil increased as the virus concentration applied increased but the percentage of the applied virus found at each depth was the same for all concentrations (Table 2). Thus, the maximum depth of penetration by the virus was independent of the concentration of applied virus.¹⁴

Virus Adsorption at Different Flow Rates

The flow rate through a coarse sand column was varied to determine the effect of flow velocity on virus adsorption. Viruses were not detected below the 160 cm depth when a coarse sand column was flooded with virus seeded sewage at a flow rate of 0.6 m/day.¹⁴ However, some viruses broke through the column when the flow rate was increased to 1.2 m/day (Table 3). Virus adsorption was not greatly affected by increasing the flow rate up to some break through point. After that break through point some viruses move through but many viruses (about 99%) were still adsorbed at a flow rate of 12 m/day. Other researchers also have re-

ported that virus movement is increased by increasing the flow rate.^{15,16} In fact, most field studies reporting virus movement through the soil to the groundwater below a land application system were located at sites with infiltration rates above 1.2 m/day.^{3,16,17} Therefore, permeability measurements may be the most important soil characteristic to use in assessing the potential for a virus break through.

Virus Detection in Field Soils

Soil samples were examined to determine if the distribution of viruses in field soils was similar to that in soil columns. When viruses were eluted from soil samples taken from groundwater recharge basins, the greatest amount of virus was found in the upper 2.5 cm of soil.¹⁸ The virus concentration in the top 2.5 cm layer were about 10 to 15 times greater than those found in the 2.5 to 10 cm layer (Table 4). This indicates that the virus adsorption pattern of the soil columns accurately represented the field adsorption pattern. About 75% of the viruses in the secondary effluent recharge basin were inactivated within 3 days after the flooding period ended. An extensive study of virus survival in the field and in laboratory soil samples is in progress.

Virus Adsorption From Different Solutions

Batch studies with small soil samples were used to study virus adsorption from different solutions and to examine adsorption by other soils. The twelve virus types and strains that showed varying degrees of adsorption to FM soil when suspended in deionized water were suspended in secondary effluent, soil extract, 0.001 M and 0.01 M CaCl_2 and added to the FM soil.¹⁹ Poliovirus adsorption by the FM soil was high (>95%) for every suspending medium (Table 5). Usually the adsorption of Echo 7 and 12 and Coxsackie B3 was over 75%. However, adsorption of Echo 1 and 29 was usually less than 50% for all of the solutions except in 0.01 CaCl_2 .

An increase in salt content has been shown to increase virus adsorption and divalent cations were more effective than monovalent cations.²⁰ Thus, the salt effect probably explains the effectiveness of the 0.01 CaCl_2 solution. The sewage water should have stimulated more adsorption than the deionized water since it contained about 600 ppm of various salts. In general, adsorption from the deionized water and sewage water were about equal indicating that something in the sewage water (possibly organic compounds) must have counteracted the salt effect. Differences in adsorption between different strains of the same virus type might result from variability in the configuration of proteins in the outer capsid of the virus since this influences the net charge on the virus.¹⁹

Virus Adsorption by Different Soils

The adsorption of the nine virus types and strains to nine different soils was compared using suspensions in deionized water (Table 6). Much variability among types and strains of viruses was observed again along with considerable variability in the adsorption capacity of different soils. In general, some viruses (like poliovirus) were adsorbed to all of the soils and some soils adsorbed all of the viruses. The soils with pH below 5.0 (B and D) had a high adsorption rate for all the viruses. Soil pH was the only soil characteristic that had a consistent effect on adsorption. Burge and Enkiri²¹ reported that bacteriophage adsorption to soils correlated with soil cation exchange capacity and surface area but no clear correlation between these factors and virus adsorption was evident in our studies.

Adsorption of Different Viruses by Soil Columns

Different viruses were applied to the soil columns to determine if the adsorption of poliovirus was comparable to adsorption of other viruses and to determine if batch studies with small soil samples could be used to predict virus movement through a soil profile. The adsorption of Echo 1 and Echo 29, two reference strains that adsorbed poorly to FM soil when added in deionized water suspension (Table 5) was compared with adsorption of poliovirus type 1 in the same soil column. The adsorption patterns of Echo 29 and Polio 1 were almost identical with 90% adsorption in the top 2 cm and no viruses detected below 160 cm (Figure 2). Adsorption of Echo 1 was 77% for the top 2 cm but all of the adsorption patterns were similar below the 40 cm depth. Thus, more Echo 1 than Polio 1 viruses moved to the 40 cm depth but numbers moving to lower depths were similar. Therefore, the potential for movement through a soil to groundwater would be similar for the two viruses if the water table was deeper than 40 cm below the soil surface. The desorption and movement of the polio and echovirus were similar when the column was flooded with deionized water (Table 7).

When adsorption data from the batch studies were compared with soil column adsorption data, clearly the adsorption studies with the 0.01 M CaCl_2 solution were the only batch tests that accurately assessed the potential for virus adsorption by the soil column. These tests showed that adsorption of Echo 29 and Polio 1 were similar and slightly more than the adsorption of Echo 1 (Table 5). Batch tests with all of the other solutions showed poor adsorption of the two Echo virus strains. However virus adsorption by soil columns was tested for only soil, the FM soil, and the 0.01 M CaCl_2 solution batch might not accurately assess the adsorption potential of columns packed with other soils. The Ca content of the sewage water increased as it moved through the calcareous FM soil as the CaCO_3 in the soil dissolved.²² This may at least partly explain why virus adsorption from suspensions of virus in 0.01 CaCl_2 were comparable with virus adsorption by a column of FM soil. Further research on adsorption of viruses by columns packed with other soils is in progress.

SUMMARY AND CONCLUSION

Most of the viruses were adsorbed near the surface when soil columns packed with Flushing Meadows sand were flooded with sewage water seeded with poliovirus. Flooding with deionized water desorbed viruses and caused some virus movement when the deionized water was applied immediately after application of the virus seeded sewage (without allowing the soil to drain before applying deionized water). When a pulse of deionized water was applied to simulate a heavy rain, the viruses moved downward but did not move through the outlet of the 250 cm long column. Thus virus would be moved by rainfall in the field only if rain fell during the first few hours of the dry period and even then viruses would not move to the groundwater if the water table was at least 250 cm below the surface. Virus movement could be prevented by resuming application of sewage water during a rain or by adding salts to the soils.

Virus adsorption from primary sewage effluent resembled that from secondary effluent. Field samples from different recharge basins flooded with primary effluent and with secondary effluent showed that most of the viruses were concentrated near the soil surface and that the adsorption pattern exhibited by soil columns accurately represented the adsorption pattern in the field.

The reason for the concentration of viruses near the soil surface is not clear. Flooding a freshly packed column with virus seeded sewage produced the same adsorption pattern and showed that it was not due to a build up of organic compounds or other materials near the soil surface. Flooding a soil column at different times with sewage effluent containing different virus concentrations showed that the number of viruses moving to a particular depth increased as the concentration of applied virus increased but the percent removal at each depth did not change with concentration. Thus the adsorption pattern was not affected by virus concentration. This means that the number of viruses reaching a particular soil depth could be reduced by sewage pretreatment (like chlorination) to reduce virus concentrations but the maximum potential depth of virus penetration would not be reduced. We hypothesized that differences in the strength of the negative charge among members of a given viral population could account for the adsorption of some viruses near the soil surface while others move deeper into the soil.

Viruses with a lower net negative charge could be adsorbed near the surface whereas those with higher net negative charges would move farther down the column. This could explain the adsorption pattern (if most of the polioviruses had a low enough net negative charge to adsorb near the surface) and could explain why the percent removal at different depths was the same when different virus concentrations were applied.

Differences in adsorption between different virus strains could also be due to differences in net negative charge among the strains. In batch studies there was a considerable difference in the degree of adsorption of different viruses to the FM soil. Most of the differences in adsorption between different viruses to FM soil were eliminated by suspending them in 0.01M CaCl₂ rather than deionized water. This might be due to the compression of the double layer or "cloud" of ions around the soil particles and viruses which allows them to move close together and be bound by London-Van der Waals forces. Also differences in adsorption by different soils were considerable. Polioviruses adsorbed to all of the soils and the soils with a pH of 5.0 or below adsorbed all of the different viruses. Soil pH was the only soil characteristic that seemed to be correlated with the degree of virus adsorption.

Some question remains about how much of the batch study data is applicable to actual situations where sewage water containing viruses moves through the soil. When a soil column (FM soil) was flooded at different times with sewage water containing either poliovirus type 1, Echovirus type 1 or Echovirus type 29, the adsorption patterns were almost identical for Polio 1 and Echo 29. The pattern for Echo 1 differed only for the upper 40 cm of soil. Only batch suspensions in 0.01 CaCl₂ had shown high adsorption rates for all three viruses. Thus the data from batch suspensions with 0.01 CaCl₂ appears to be applicable to virus adsorption by a FM soil profile but may not be applicable for other soils with a lower salt content.

Also the flow rate through soil columns or a field soil can affect virus movement. The flow rate through the soil columns did not affect virus adsorption until the rate was increased above a breakpoint of about 120 cm/day. Virus broke through the columns at rates higher than 120 cm/day. Thus, a measure of soil permeability may be the single most valuable test to use in assessing the potential for virus movement through a soil.

Further research on movement and survival of viruses in columns packed with different soils is in progress.

ACKNOWLEDGEMENT

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TABLE 1
Major Characteristics of Soils*

Sample Code	Soil Series	Family	Surface Area m ² /g	Clay per cent	Silt per cent	Sand per cent	Cation Exchange Capacity meg/l	Organic Matter per cent	Saturated pH
B	Vernon	Fine, mixed, thermic	84	39	13	48	32	0.3	4.5
C	Clarita	Fine, montmorillonitic, thermic	203	54	20	26	71	4.2	7.1
D	Windthorst	Fine, mixed, thermic	155	53	16	13	53	1.4	4.9
F	Chigley	Fine, mixed, thermic	52	28	13	59	23	1.4	8.0
H	--	Not classified	105	36	24	40	30	0.78	8.0
K	--	Sandy, siliceous, hyperthermic	--	3	8	89	6.5	3.64	7.1
T	Anthony	Coarse-loamy, mixed (calcareous), thermic	38	13	10	77	4.2	0.27	8.2
X	Rubicon	Sandy, mixed, frigid	18.4	4	4	92	5.6	0.4	5.5
FM	--	Loamy sand	--	3	8	89	--	--	7.8

*For additional characteristics, see Enfield et al. 1976.

TABLE 2
Percentage of added viruses remaining at various depths after flooding soil columns with sewage water containing different concentrations of poliovirus.

Column Depth cm	Added Virus Concentration PFU/ml		
	1×10^2	3.9×10^3	2.6×10^4
	Per	Cent	Virus Remaining*
2	35	23	18
5	26	19	22
10	7	5	6
20	6	4	5
40	8	5	5
80	4	0.1	0.5
160	0.7	0	0
240	0	0	0
250	0	0	0

*Data are averages for samples taken on 3 flooding days.

TABLE 3
Virus removal at various depths in a soil column flooded at different infiltration rates with sewage effluent seeded with poliovirus.

Column Depth cm	Flow rate m/day				
	0.6	1.2	2.4	4.0	12.0
	Per	Cent	Virus	Remaining	
2*	71	86	46	39	54
5	55	71	31	31	36
10	32	67	74	23	32
20	13	45	77	21	26
40	15	43	51	10	12
80	2	15	24	4	15
160	1	5	9	1	3
240	0	3	6	2	1
250	0	2	4	1	--

*Samples were taken 1 day after application of virus for the first three flow rates and 6 to 9 hours after application for the two highest rates. Virus concentrations at 0 depth ranged from 5600 to 28000 PFU/ml.

TABLE 4
Viruses eluted from soils in ground water recharge basins flooded with sewage water.

Soil Depth cm	Primary Effluent Basin	Secondary Effluent Basin	
	PFU/200 g Soil	PFU/200 g Soil	
	Day 1	Day 1	Day 3
0 - 2.5	101	35	6
2.5 - 10	6	3	2
10 - 25	5	1	1

TABLE 5
Effect of Suspending Medium on Virus Adsorption to FM Soil

Virus Type	Deionized Water		Secondary Effluent		Virus Adsorption With:		0.001 M CaCl ₂		0.01 M CaCl ₂	
	Per Cent		Per Cent		Soil Extract	Per Cent	Per Cent		Per Cent	
<u>Reference Strains:</u>										
Polio 1 LSC	99.9		99.3		96		96		99.0	
Echo 1	55		0		38		4		82	
Echo 7	99.9		94		97		96		97	
Echo 12	78		84		97		14		99.0	
Echo 29	14		12		54		23		99.2	
Coxsackie B3	99.4		93		87		72		99.0	
<u>Recent Isolates:</u>										
Echo 1 V212	46		0		0		0		78	
Echo 1 V239	0		14		0		7		80	
Echo 1 V248	30		12		0		7		75	
Echo 1 V249	35		6		0		34		70	
Coxsackie B4 V216	30		27		6		15		85	
Coxsackie B4 V240	0		17		37		0		80	

TABLE 6
Adsorption of Enteroviruses to Different Soil Types

Soil Types	Virus Adsorption									
	Reference Strains				Recent Isolates					
	Polio 1 LSc		Echo 1		Echo 7		Coxsackie B3		Echo 1	
	Per Cent	Per Cent	Per Cent	Per Cent	Per Cent	Per Cent	Per Cent	Per Cent	Per Cent	Per Cent
B	99.9	99.7	99.0	99.0	99.6	99.7	98	99.8	99.3	98.7
C	95	21	98	98	92	23	10	25	2	0
D	94	90	68	68	83	70	65	99.9	67	92
F	99.0	11	99.0	99.0	97	19	53	33	32	26
H	99.6	12	99.5	99.5	97	38	49	25	34	24
K	42	28	20	20	56	44	5	0	12	56
T	82	13	22	22	35	1	52	33	34	35
X	56	78	8	8	73	79	68	82	44	73
FM	99.9	60	99.9	99.9	97	32	22	16	19	97

TABLE 7

Virus Concentration PFU/ml After One Day of Flooding With Dionized Water

Depth cm	Column 2	Column 4
	Echo 1 Virus	Polio Virus
0	200	450
2	400	360
5	1300	1200
10	300	210
20	200	90
40	150	240
80	55	250
160	15	15
250	0	0

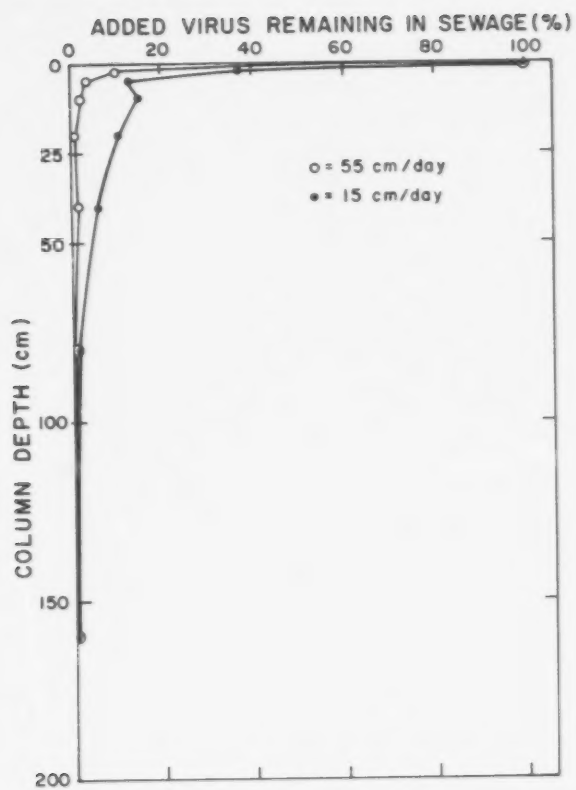


Fig. 1 Poliovirus removal from sewage water by soil columns with different infiltration rates. Added concentration of virus was 3 to 5×10^6 PFU/ml.

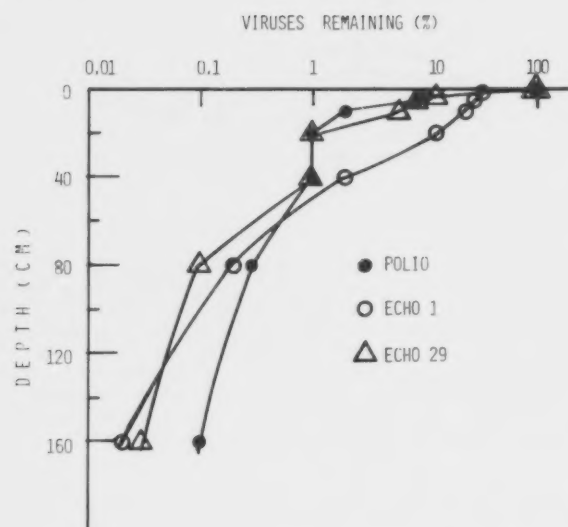


Fig. 2. Adsorption of different viruses by a soil column.

BEHAVIOR OF VOLATILE AND EXTRACTABLE ORGANICS IN COMBINED
BIOLOGICAL/PHYSICAL-CHEMICAL TREATMENT OF MUNICIPAL WASTEWATER

Thomas A. Pressley
Research Chemist
U.S. Environmental Protection Agency
Municipal Environmental Research Laboratory
Wastewater Research Division
Cincinnati, Ohio 45268

INTRODUCTION

The increasing pollution of existing fresh water sources, as well as the need for wastewater reuse in many parts of the world, is focusing attention on more efficient methods of wastewater treatment. Effluents from the more common biological treatment processes contain considerable amounts of organic materials which are responsible for taste, odor, and color, some of which may possess the potential for severe toxic effects on the biota of freshwater bodies. In wastewater reuse for human consumption, considerable attention has been given to aesthetic, as well as physiological effects of the organic residuals thus far identified (1).

The organic materials in wastewaters affect, and are affected by, many of the physical-chemical and biological processes in use today. A better understanding of the composition and characteristics of the organic materials in raw wastewater, as well as effluents after various stages of treatment aid in better design and operation of wastewater treatment processes. The purpose of this study was to examine qualitatively and semi-quantitatively the behavior of the volatile (purgeable) organic materials and the semi-volatile methylene chloride extractable organic materials after the major steps of treatment in a combined biological/physical-chemical treatment reuse process.

In the physical-chemical treatment of raw wastewater, lime and a mineral salt such as $MgSO_4$, $FeCl_3$ or alum are added to the wastewater in the reactor to attain a pH of 10.5-11.6. Under these conditions, bicarbonate ions are converted to carbonate ions and precipitated by the excess calcium ions as calcium carbonate. Phosphorous is precipitated as calcium hydroxyapatite with soluble residuals below 0.1 mg/l (as P). The magnesium or ferric ions, converted to their gelatinous hydroxides, provide very efficient flocculation and sedimentation of the calcium carbonate, calcium hydroxyapatite and particulate organic matter in the clarifier following the reactor. Following lime clarification and sedimentation, ammonia removal is achieved by biological nitrification-denitrification or breakpoint chlorination. The wastewater is then treated with activated carbon (either granular or powdered) for the removal of dissolved organic materials. In some cases, effluents from carbon adsorption are "polished" by breakpoint chlorination for residual ammonia removal and disinfection.

In the combined biological/physical-chemical-treatment process examined in these studies, approximately 200 mg/l of CaO and 15 mg/l $FeCl_3$ (as Fe) were added for lime clarification, at pH 10.5 (low pH-lime process). Nitrification (suspended growth) and denitrification (fixed film) were employed for ammonia removal. Methanol dosages corresponding

to methanol:NO₃-N ratios of 2:1 to 4:1 were used in denitrification. Dissolved organic material was removed by passing the effluent from the denitrification stage through granular (8 x 30 mesh) activated carbon (Filtrisorb 300) contained in four columns in series at a 4.8 l/m²s loading rate (downflow packed bed operation). Following carbon adsorption, the wastewater was treated with approximately 5 mg/l of alum as Al and passed at approximately 2 l/m²s hydraulic loading rate, through a dual media filter, containing 0.6 meter of coal and 0.3 meter of sand. This final flocculation and filtration step was designed to remove particulate matter formed by, or escaping from, the carbon adsorption stage. The wastewater was then chlorinated to achieve a free available chlorine concentration of 1 mg/l for removal of any residual ammonia and for disinfection. The physical-chemical treatment process required a total detention time of 10.6 hours at a flow rate of 35 gpm (2.2 x 10⁻³ m³/s).

EXPERIMENTAL

Wastewater samples used in these studies were taken at the U.S. Environmental Protection Agency-District of Columbia Pilot Plant located at 5000 Overlook Avenue, S.W., Washington, D.C. The pilot plant was designed for research in the development and demonstration of more efficient methods of municipal wastewater treatment. The combined biological/physical-chemical treatment process examined in this study is shown schematically in figure 1. Chemical parameters typical of the District of Columbia raw wastewater and effluents after various stages of treatment are shown in Table 1.

Wastewater grab samples were taken after the following stages within the combined biological/physical-chemical treatment process:

- Raw Wastewater (H-1)
- Denitrification (I-7)
- Carbon Adsorption (J-7)
- Alum Coagulation and Filtration (K-7)
- Chlorination and Filtration (Final Effluent) (L-7)

Samples for volatile organic analysis (VOA) were taken in 15 ml crimp-seal bottles containing a film of Na₂S₂O₃ sufficient to reduce the total available chlorine in a 15 ml aliquot of water containing 30 mg/l of total available chlorine. The bottles were completely filled to avoid headspace losses and crimp-sealed with a teflon seal. Volatile organic materials were determined by GC/MS on a 5 ml sample employing the purge and trap procedures and apparatus described by Bellar and Lichtenberg (2). The trap used in these tests was filled one third with silica gel (Davidson 15, 35/60 mesh) and two thirds with Tenax-GC so that the purged volatile organic materials were exposed to the Tenax-GC first. The purged volatile organic materials adsorbed on the trap were thermally desorbed for 3 minutes in the modified injection port of the Finnigan 9500 gas chromatograph at 200°C and with fifteen cc/min helium flow, on to the chromatographic column held at ambient temperature.

Following the 3-minute thermal desorption in the injection port, the trap was removed from the injection port of the GC. The injection port

was immediately plugged with the prescribed quick-connect plug and the trap baked out at 200°C for 20 minutes under 15 cc/min helium flow. The trap was then plugged and allowed to cool to ambient temperature before reuse. The gas chromatograph was equipped with a 5' glass column of 1/4" OD (2 mm ID) and packed with Tenax-GC, 60/80 mesh (Supelco, Inc.). The injector port was maintained at 200°C for sample vaporization and thermal desorption of the Tenax-GC-silica gel trap. The separator oven and transfer lines were maintained at 250°. Temperature programming in the VOA involved maintaining the chromatographic column at ambient temperature during thermal desorption of the trap, until the water vapor eluted from the chromatographic column. The water vapor was observed as a pressure surge in the analyzer of the mass spectrometer--its elution required approximately two minutes. An additional two minutes were allotted at ambient temperature before temperature programming to 190°C at 8 deg/min. The final temperature was maintained for 10 minutes before ending data acquisition. Data acquisition on the Finnigan 3300 Mass Spectrometer continued following elution of the water vapor from the chromatographic column for a total data acquisition time of 32 minutes.

A Finnigan 3300 mass spectrometer was utilized in these studies to provide mass spectra of the components separated by the gas chromatograph. The Finnigan 3300 GC/MS system was interfaced to a Systems 150 GC/MS Data System (PDP-8) for rapid processing of mass spectral data.

The GC/MS system was operated in the control mode over a mass range of 30-200, 201-400 amu, using integration times of 8 ms/amu, for a 3.8 second scan time for the VOA. Sensitivity was set at 10⁷ amps/volt and electron energy was 70 ev. Real time attenuation was set so that 750 ng of chloroform produced a full scale response in order to have a working range of 1 to 150 ppb chloroform.

Grab samples taken for the analysis of methylene chloride extractable acidic, neutral, and basic organic materials were contained in 1 gallon wide-mouth glass jars under an aluminum foil seal. The jars were filled in order to avoid head space losses. The samples were not dechlorinated. The samples were packed in ice and shipped by air express to the Cincinnati laboratory for extraction. The time interval between sample selection and sample extraction was 24-48 hours. The samples, usually 4-liter aliquots, were extracted in 6-liter separatory funnels with methylene chloride for acidic, neutral, and basic materials. The methylene chloride extracts were dried, concentrated, and the solvent changed to acetone according to the procedure described in the EPA GC/MS Procedural Manual (3). The final acidic, neutral, and basic methylene chloride extractable organic materials (now in acetone), were concentrated, under a gentle flow of dry nitrogen, to 1.0 ml and held at 4°C for GC/MS analysis.

The Finnigan 9500 gas chromatograph used in the analysis of the acidic, neutral, and basic extracts was equipped with a 30-meter glass capillary column coated with SE-30 and possessing 86,000 effective theoretical plates.

The gas chromatograph was modified to allow the flow rate of the helium carrier gas to be controlled by adjusting the pressure regulator at the main cylinder rather than at the controls of the gas chromatograph. The Finnigan GC/MS system was further modified for these studies by installing a glass-lined transfer line to deliver the column effluent

directly to the analysis of the mass spectrometer, by-passing the separator.

The helium pressure was adjusted to approximately 7 psi and the Grob injector adjusted to yield a linear velocity of 30 cm/sec through the column at 100°C with pentane, and a split ratio of approximately 30:1. The injector and transfer lines were maintained at 270°C. The Grob injector was closed for at least 30 seconds prior to sample injection, and opened 30 seconds after injection of the sample.

Temperature programming selected for the analysis of the acidic, neutral, and basic extracts started at approximately 30°C (ambient temperature) and was programmed to 250°C at 4 degrees per minute. The final temperature was maintained for 15 minutes for a total data acquisition run time of 70 minutes. Temperature programming and data acquisition were begun 3 minutes after sample injection in order to avoid data acquisition during elution of the solvent.

Additional mass spectrometric conditions employed in the analysis of the acidic, neutral, and basic extracts included operation in the control-mode over the mass range of 20-200, and 201-500, with integration times of 2 and 4 ms/amu respectively. The electron energy was set at 70 eV and the emission current was 0.5 amp. Amplifier sensitivity was set at 10^{-7} amps/volt. The ion energy, extractor, and lens voltages were adjusted for optimum peak, shape and sensitivity.

QUALITY ASSURANCE

Quality assurance measures during the VOA involved achieving ion abundance data from perfluorotributylamine, which matched that taken when the mass spectrometer was properly tuned and verified from ion abundance data from decafluorotriphenylphosphine. Additional sensitivity and chromatographic performance checks were made by running fresh standard solutions.

Quality assurance measures throughout the analysis of the acidic, neutral, and basic extracts involved strict compliance with the GC/MS tuning and sensitivity recommendations prescribed by Eichelberger and Budde (4) with decafluorotriphenylphosphine under identical sample analysis conditions. Additional column performance and sensitivity checks were made by running a C₁₈, C₁₉, and C₂₀ hydrocarbon mixture. Sensitivity was achieved so that 10 ng of the hydrocarbons produced a 4:1 signal to noise ratio.

Compound identifications were made by computerized spectral matching techniques and by running and matching against known standards when available. Blanks were carried through each extraction process and compensations were made during data analysis. All glassware exposed to the sample prior to GC/MS analysis was previously cleaned by being washed with hot detergent solution, rinsed with tap water, rinsed with distilled water, air dried and heated in a muffle furnace at 400°C for 30 minutes.

The methylene chloride and acetone solvents used in these tests were "distilled in glass" quality obtained from Burdick and Jackson and were used as received. The anhydrous sodium sulfate for drying the extracts was analytical reagent quality, obtained from Fisher Scientific Co., and was used as received.

DISCUSSION OF RESULTS

A. Methylene Chloride Extractable Organics

The complexity of the total ion chromatograms of the extracts of methylene chloride extractable materials from wastewater after varying stages of physical-chemical treatment seemed to parallel the TOC and COD values (Table 2). The total ion chromatograms of the various extracts are presented to demonstrate effluent quality and are typical (Fig. 3-15). The methylene chloride extractable materials in the raw wastewaters (H-1) showed considerable variation in content as well as in complexity of the total ion chromatograms. No single organic compound was found in the 8/3/76 raw wastewater extracts above 10 ppb (Table 3). In the 11/4/76 raw wastewater extracts, only stearic and palmitic acids were found at concentrations above 10 ppb. These acids, as well as lauric and oleic were found in the neutral fraction and were not found at concentrations above 1 ppb in the acidic fraction. Caffeine was also found in the neutral fraction and not detected in the basic fraction (Table 4). The large number of saturated and unsaturated aliphatic hydrocarbons detected in the raw wastewater extracts indicate motor oil and other internal combustion engine by-products. No organic bases were detected in the raw wastewater extracts except caffeine and a trace of a compound tentatively identified as N,N-diphenylhydrazine (Table 4).

Following lime precipitation (clarification), biological nitrification and denitrification, (I-7), the methylene-chloride extractable organic materials in the wastewater were dramatically reduced from that of the raw wastewater (Tables 5 and 6) (Fig. 7-9). Only acetic acid was identified in the neutral fraction of I-7 at a concentration greater than one ppb (Table 6). This probably was sample contamination since acetic acid is sufficiently volatile to have been lost during Kuderna-Danish concentration and should not have been extracted into the neutral fraction.

Following carbon adsorption, GC/MS analysis of the K-7 extracts revealed further reduction of complexity in the total ion chromatograms (Fig. 10-12). The TOC and COD analyses revealed a parallel reduction of total organic matter. No compounds were identified in any of the K-7 extracts at concentrations greater than 1 ppb (Tables 7 and 8). Likewise, no peaks were observed in the total ion chromatograms of the K-7 extracts that could have represented concentrations greater than 1 ppb.

Following chlorination for residual ammonia removal and disinfection and alum addition and filtration for residual turbidity removal, the total ion chromatograms of the final effluent (L-7) extracts revealed no significant difference from those of the K-7 extracts (Tables 9 and 10) (Fig. 13-15). All of the compounds identified in the L-7 extracts were at the low ppb levels (1-5 ppb) or at the lower limits of detection by GC/MS under the conditions employed in these tests. These low concentrations encountered in the methylene chloride extracts of the final effluents from the wastewater treatment system made spectral identification difficult and was responsible for most of the tentative identifications made.

Considerable quantities of chlorinated and brominated cyclohexanes, cyclohexenes, and cyclohexanols were detected in the neutral and acidic extracts of the final effluent (L-7) (Fig. 13 and 14). The presence of

these compounds was attributed to the extraction of wastewater containing free chlorine residuals with methylene chloride preserved with cyclohexene (5).

B. Volatile (Purgeable) Organics

Halogenated ethanes and methanes were compiled into a separate table here for special discussion due to the recent emphasis placed on these materials as possible carcinogens.

In the Washington, D.C. raw wastewater, chloroform concentrations averaged a rather steady 10 $\mu\text{g/l}$ in the summertime grab samples. Other halogenated methanes and ethanes identified in the raw wastewater were at concentrations below 10 $\mu\text{g/l}$ (Table 11). The concentration of chloroform in the wastewater was reduced 50 to 100 percent following biological nitrification and denitrification (I-7). The concentrations of other halogenated methanes and ethanes identified in the wastewater following biological nitrification and denitrification were reduced to levels not detectable by the conditions employed in these tasks. This reduction in the concentration of volatile halogenated methanes and ethanes was attributed to the purging action of the aeration step incorporated in the nitrification step previously described (Fig. 1). No further reductions in the volatile halogenated methanes and ethanes were observed following the carbon adsorption (J-7). Indeed, chloroform concentrations may have increased slightly following carbon adsorption as shown in the 7/1/76 to 7/8/76 tests.

After chlorination to a free chlorine residual of approximately 1 mg/l, the final effluent (L-7) contained concentrations of chloroform ranging from 5 to 10 $\mu\text{g/l}$. Other halogenated methane and ethane concentrations increased from undetectable levels to levels ranging from 3 to 5 $\mu\text{g/l}$ (Fig. 16). More data is necessary to substantiate this apparent formation of halogenated methanes and ethanes by chlorination of treated wastewaters, because in these studies, no attempt was made to ascertain that the chlorine used in the chlorination step was not contaminated with chloroform and other volatile halogenated materials.

Other volatile (purgeable)/organic materials identified in the wastewaters are tabulated in Table 12. Organic compounds other than the halogenated methanes and ethanes were not consistently detected in the final effluent (L-7) above 1 $\mu\text{g/l}$.

CONCLUSIONS

The dramatic reduction in the total organic material in the wastewater following lime clarification and biological nitrification and denitrification, as monitored by COD and TOC paralleled the reduction in the complexity and number of peaks in the respective gas chromatograms (Fig. 2). The efficient removal of the specific organic matter by lime precipitation and biological nitrification and denitrification (ammonia removal step) suggested effective:

- a. Adsorption and absorption by the gelatinous hydroxides formed during the lime clarification step, with subsequent removal by sedimentation (occlusion).

- b. Purging of the more volatile organic materials from the water during the aeration step of biological nitrification.
- c. Biological degradation of the organic materials and/or adsorption into the bio-mass during the nitrification and denitrification processes.

The activated carbon process reduced the wastewater TOC from approximately 6 mg/l to 2 mg/l. The absence of peaks in the gas chromatograms of the acidic, neutral, and basic fractions of the extracts from treated wastewater samples taken after the activated carbon treatment step indicated that the residual organic materials (~ 2 mg/l of TOC) may be of amphoteric nature and thus not extractable by organic solvents used in analytical methods, or that any specific extractable organic may be at concentration levels below the level of detection employed in these tests. Further work is needed to characterize the 2 mg/l of residual TOC in the renovated water.

The results of these studies of the behavior of volatile and extractable organics in a combined biological/physical-chemical treatment of a municipal raw wastewater revealed final effluent concentrations of halogenated methanes and ethanes, other volatile organic materials, and the extractable materials to be similar to those found in finished drinking waters during the EPA National Reconnaissance Survey of 1975. The specific organic compounds identified in the final effluent have also been identified in finished drinking waters (6).

A portion of the VOA data generated in this study has been presented in a previous report (6). It is suggested that this reference be consulted for a detailed comparison of the reuse effluent with finished drinking waters.

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TABLE 1

Chemical Parameters Typical of the District of Columbia
Raw Wastewater and After Various Stages in the
Combined Biological/Physical-Chemical Treatment Process

Chemical Parameters	H-1	I-7	K-7	L-7
Alkalinity, P	-	-	-	-
Alkalinity, MO	126	-	-	96
pH	7.2	-	7.5	7.5
Conductivity (MHOS)	-	-	-	710,000
TOC	72	7.0	2.5	2.7
BOD	104	4.5	1.3	2.8
COD	237	18.5	6.6	6.5
Total P (PO_4)	15.0	0.3	0.17	0.16
TKN	19.0	0.76	0.25	0.22
$\text{NH}_3\text{-N}$	16.0	0.20	0.95	0.65
$\text{NO}_2^- + \text{NO}_3^-\text{-N}$	0.89	4.7	4.8	4.8
Suspended Solids	107	3.9	0.97	0.86
VSS	83.0	2.7	-	-
TS	-	-	-	368
Ca^{++}	32	-	-	56
T-Fe	1.3	-	-	0.6
Mg^{++}	6.5	-	-	5.4
Cl^-	-	-	-	68.7
SO_4^{--}	-	-	-	50.7
Na^+	-	-	-	34.1
K^+	-	-	-	8.2
F^-	0.7	-	-	0.7
MBAS	8.9	.28	-	.14
Al^{+++}	-	-	.25	-

TABLE 2
Organic Materials Identified in the 8/3/76
Raw Wastewater Extract

Compound	Concentration (µg/l)
<u>H-1 Neutrals</u>	
Tetrachloroethylene	3
Methylethylbenzene	1
Trimethylbenzene	1
Trimethylbenzene	1
Saturated Aliphatic Hydrocarbon	1
Aromatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
<u>H-1 Acids</u>	
Caprylic Acid	1
Octanoic Acid	1
Nonanoic Acid	1
Decanoic Acid (Tentative)	1
Palmitic Acid	1
Oleic Acid	2
Stearic Acid	2
<u>H-1 Bases</u>	
Caffein	1

TABLE 3
Organic Materials Identified in the 11/4/76
Raw Wastewater Extract

Compound	Concentration (µg/l)
<u>H-1 Neutrals</u>	
Tetrachloroethylene	2
1,3-Xylene	1
2-Butoxyethane	2
Diocylether (Tentative)	1
1-Methylethylbenzene	1
?	1
Trimethylbenzene	1
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
Trimethylbenzene	2
Methyl-isopropylcyclohexane	1
?	2
Saturated Aliphatic Hydrocarbon	1
?	3
?	3
?	1
Alpha Terpinol	1
Alpha Terpinol	2
?	3
?	3
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	2
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	2
Saturated Aliphatic Hydrocarbon	4
Saturated Aliphatic Hydrocarbon	2
1,3-dimethylnaphthalene	4
Saturated Aliphatic Hydrocarbon	2
Saturated Aliphatic Hydrocarbon	2
Trimethylnaphthalene	4
Saturated Aliphatic Hydrocarbon	1
Aliphatic Ketone	1
Saturated Aliphatic Hydrocarbon	2
Saturated Aliphatic Hydrocarbon	3
Lauric Acid	2
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	2
Saturated Aliphatic Hydrocarbon	1
Oleic Acid	3
Caffein	1
Saturated Hydrocarbon	1
Palmitic Acid	> 20
Saturated Hydrocarbon	-
Stearic Acid	> 20
?	2
Saturated Hydrocarbon	1
Saturated Hydrocarbon	1
Steroid	1
Steroid	1

TABLE 3 (Cont'd)

Organic Materials Identified in the 11/4/76
Raw Wastewater Extract

Compound	Concentration (ug/l)
<u>H-1 Acids</u>	
3-methyl pentanal	1
Nitroethylpropionate (Tentative)	1
2-N-butoxyethanol	1
Hexanoic Acid (Tentative)	1
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	2
Saturated Aliphatic Hydrocarbon	3
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
Unsaturated Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	3
2-Undecenal (Tentative)	1
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon C ₈ -C ₁₂	3
Saturated Aliphatic Hydrocarbon C ₁₂ -C ₁₄	2
Saturated Aliphatic Hydrocarbon	3
Saturated Aliphatic Hydrocarbon C ₈ -C ₁₃	2
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon C ₁₂ -C ₂₂	1
Unsaturated Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
Palmitic Acid	1
Di(2-ethylhexyl) phthalate	1
?	1
<u>H-1 Bases</u>	
N,N-Diphenyl hydrazine	(trace)
?	1
?	1

TABLE 4

Organic Materials Identified in the 8/3/76
I-7 Extract

Compound	Concentration ($\mu\text{g/l}$)
<u>I-7 Neutrals</u>	
5 methyl-1-hexene	1
1-octene	1
3-Nonone (Tentative)	1
3-hexanol	1
Tetraethyl lead	1
Tetramethylsilane	1
Butyl phthalate	1
<u>I-7 Acids</u>	
Saturated Aliphatic Hydrocarbon	1
Unsaturated Aliphatic Hydrocarbon	1
Benzothiazole	1
Butyl phthalate	1
<u>I-7 Bases</u>	
No peaks above 1 ppb	

TABLE 5
Organic Materials Identified in the 11/4/76
I-7 Extract

Compound	Concentration (ug/l)
<u>I-7 Neutrals</u>	
Acetic Acid	3
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
1,2-ethane diacetate	1
Aliphatic Alcohol	1
Cyclohexenone	1
Saturated Aliphatic Hydrocarbon	1
?	1
4-heptanone	1
4-chlorocyclohexanol	1
?	1
?	1
Aliphatic Hydrocarbon	1
?	1
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
4-methyl-2,6-ditert-butylphenol	1
Saturated Aliphatic Hydrocarbon	1
4-ethyl-2,6-ditert-butylphenol	1
Diethylphthalate	1
Aromatic Hydrocarbon	1
?	2
1,2-dimethyl-4-benzylbenzene	1
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
Saturated Aliphatic Hydrocarbon	1
?	1
<u>I-7 Acids</u>	
No peaks above 1 ppb	
<u>I-7 Bases</u>	
No peaks above 1 ppb	

TABLE 6
Organic Materials Identified in the
8/3/76 K-7 Extract

Compound	Concentration (µg/l)
<u>K-7 Acids</u>	
Acetic Acid	1
Unsaturated Hydrocarbon	1
Unsaturated Hydrocarbon	1
3-heptanol	1
3-heptanone	1
<u>K-7 Bases</u>	
No peaks above 1 ppb	

TABLE 7
Organic Materials Identified in the
11/4/76 K-7 Extract

Compound	Concentration (µg/l)
<u>K-7 Neutrals</u>	
Unsaturated Aliphatic Hydrocarbon	1
Unsaturated Aliphatic Hydrocarbon	1
<u>K-7 Acids</u>	
Unsaturated Hydrocarbon	1
Unsaturated Hydrocarbon	1
Di-(2-ethylhexyl) phthalate	1
"α-phthalate"	1
<u>K-7 Bases</u>	
Unsaturated Hydrocarbon	1
Unsaturated Hydrocarbon	1
N,N-diphenylhydrazine (Tentative)	1

TABLE 8

Organic Materials Identified in the 8/3/76
L-7 (Final Effluent) Extract

Compound	Concentration (µg/l)
<u>L-7 Neutrals</u>	
Unsaturated Hydrocarbon (C ₁₅ -C ₁₉)	2
Isobutyl phthalate	3
<u>L-7 Acids</u>	
No peaks above 1 ppb	
<u>L-7 Bases</u>	
5-methyl-1-hexene	1
Trans-2-octenal (Tentative)	3

TABLE 9

Organic Materials Identified in the 11/4/76
L-7 (Final Effluent) Extract

Compound	Concentration (µg/l)
<u>L-7 Neutrals</u>	
Unsaturated Aliphatic Hydrocarbon	2
Unsaturated Aliphatic Hydrocarbon	2
3-methyl-2-pentanone	1
Saturated Aliphatic Hydrocarbon	1
C ₂₀ -C ₂₂ Carboxylic Acid	2
Diisobutyl phthalate	1
2-butoxy-2-oxoethyl-butyl phthalate	4
Diisooctylphthalate	2
<u>L-7 Acids</u>	
Hydroxy-2-propanone	1
Acetic Acid	1
Unsaturated Hydrocarbon	1
Di-(2-ethylhexyl) phthalate	1
<u>L-7 Bases</u>	
No peaks from sample above 1 ppb	

TABLE 10

Halogenated Ethanes and Methanes Identified in
Effluents of the Combined Biological/Physical-Chemical Process

Compound	Extract No.			
	H-1	I-7	J-7	L-7
<u>6/18/76</u>				
Chloroform	10	5	5	7
Bromodichloromethane	-	-	-	3
Dibromochloromethane	5	-	-	1
Bromoform	-	-	-	-
Carbon tetrachloride	2	-	-	-
Tetrachloroethylene	4	-	-	-
1,1,1-Trichloroethane	-	-	-	-
<u>7/1/76</u>				
Chloroform	10	-	5	10
Bromodichloromethane	-	-	-	5
Dibromochloromethane	1	-	2	5
Bromoform	-	-	-	-
Carbon tetrachloride	4	-	-	-
Tetrachloroethylene	3	-	-	-
1,1,1-Trichloroethane	2	-	-	-
<u>7/8/76</u>				
Chloroform	10	1	6	5
Bromodichloromethane	1	Trace	-	3
Dibromochloromethane	5	-	Trace	-
Bromoform	-	-	-	-
Carbon tetrachloride	4	-	-	-
Tetrachloroethylene	3	-	-	-
1,1,1-Trichloroethane	2	-	-	-

TABLE 11

Other Volatile (Purgeable) Organic Materials
Identified in Effluents of the Combined
Biological/Physical-Chemical Process

Compound	H-1	I-7	J-7	K-7
<u>6/18/76:</u>				
Acetaldehyde	-*	✓*	✓	-
Methanol	-	-	-	-
Acetone	✓	✓	✓	✓
Dichloromethane	10	-	1	1
Acrolein	-	✓	-	✓
Carbon disulfide	-	-	-	✓
Dimethyl disulfide	6	-	-	-
Toluene	2	-	-	-
Xylene	1	-	-	✓
Alkyl benzene	1	-	-	-
Benzaldehyde	-	✓	-	-
<u>7/1/76:</u>				
Acetaldehyde	✓	-	-	-
Methanol	-	-	-	-
Acetone	✓	✓	✓	✓
Dichloromethane	1	-	1	-
Acrolein	✓	-	-	-
Carbon disulfide	✓	-	-	-
Dimethyl disulfide	-	-	-	-
Toluene	2	-	✓	-
Xylene	-	✓	✓	✓
Alkyl benzene	✓	-	-	-
Benzaldehyde	-	✓	-	5
<u>7/8/76:</u>				
Acetaldehyde	-	-	-	-
Methanol	✓	-	-	-
Acetone	✓	✓	✓	✓
Dichloromethane	-	✓	2	✓
Acrolein	-	-	-	-
Carbon disulfide	-	-	✓	-
Dimethyl disulfide	-	-	-	-
Toluene	2	✓	✓	✓
Xylene	-	✓	✓	-
Alkyl benzene	-	-	-	-
Benzaldehyde	-	-	-	✓

* Compounds represented by a dash (-) = not detected.
Compounds represented by a check (✓) = trace amounts (less than
1 µg/l.

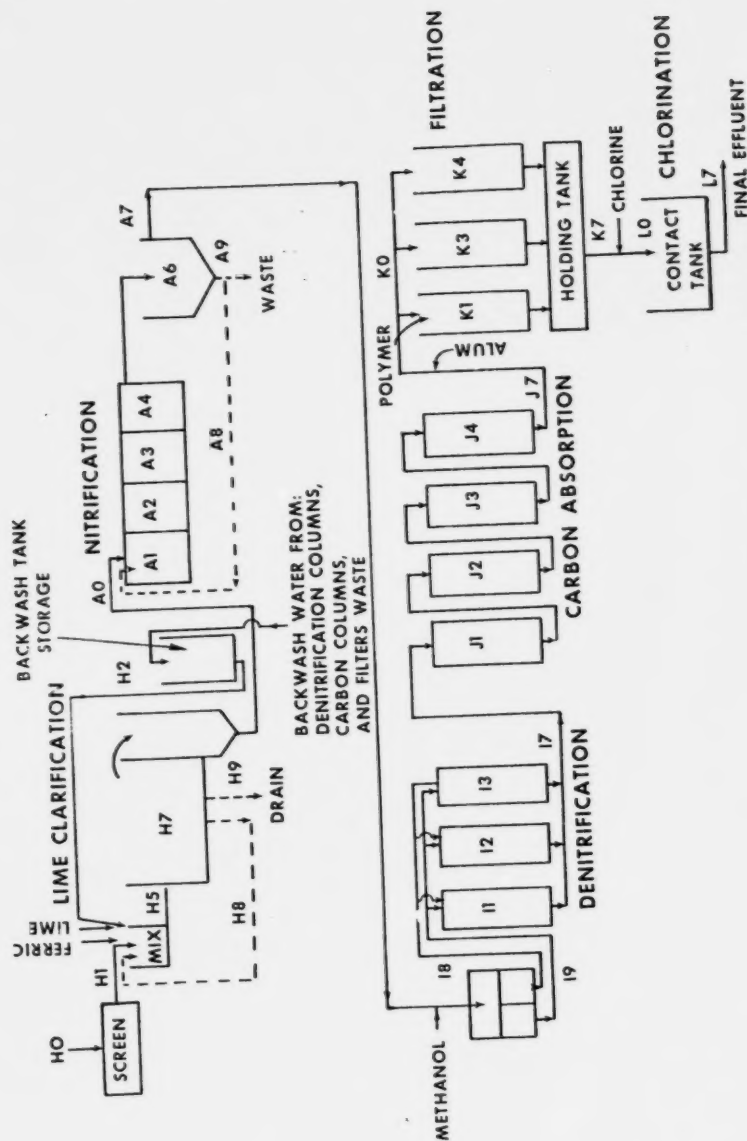


Fig. 1. Schematic Flow Diagram

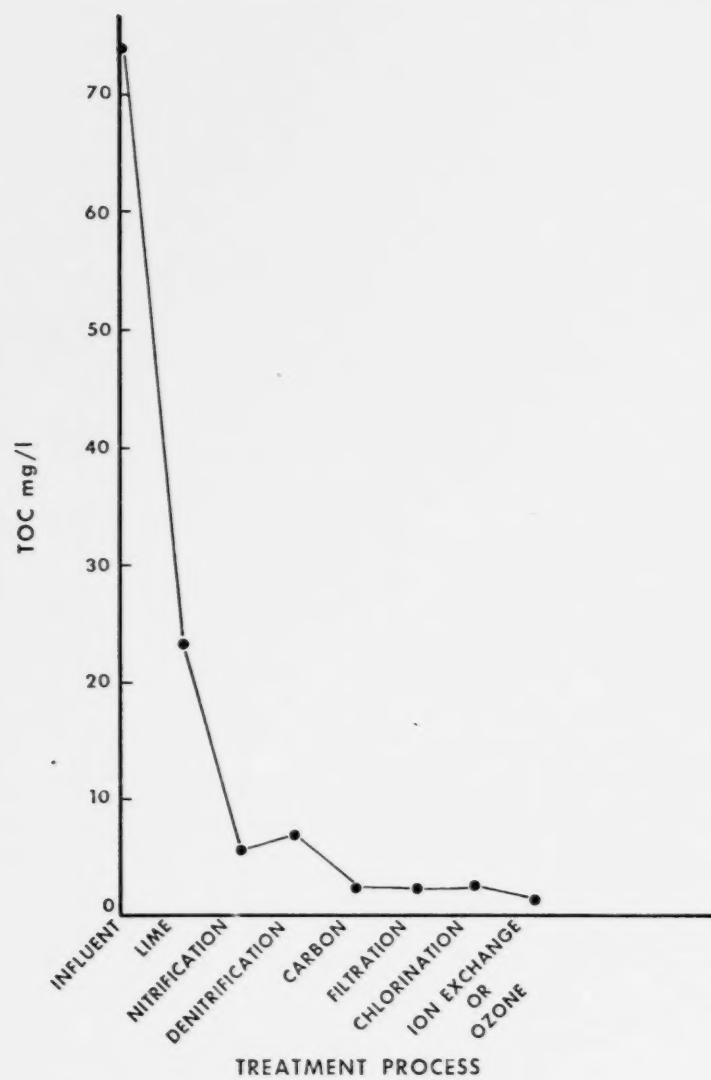


Fig. 2. Removal of TOC



Fig. 3. Total Ion Chromatogram of Blank Extract

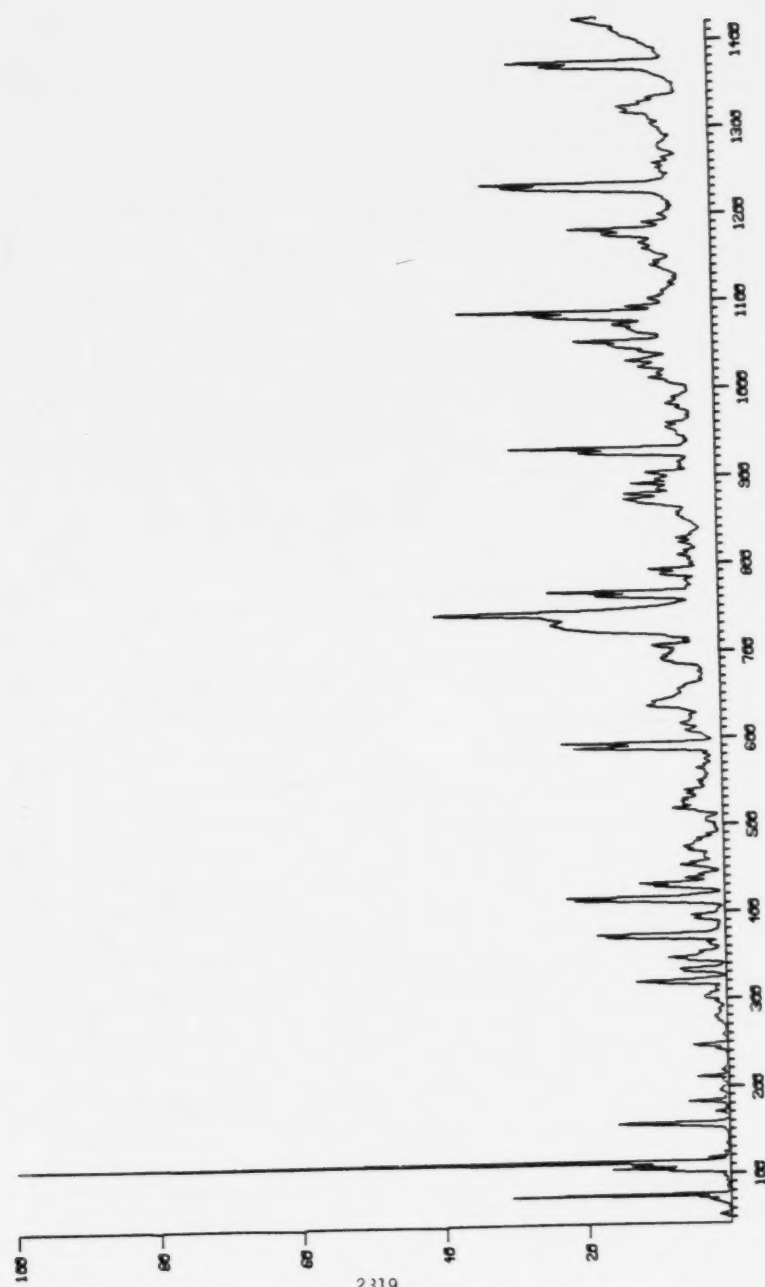


Fig. 4 Total Ion Chromatogram of 8/3/76 H-1 Neutral Extract

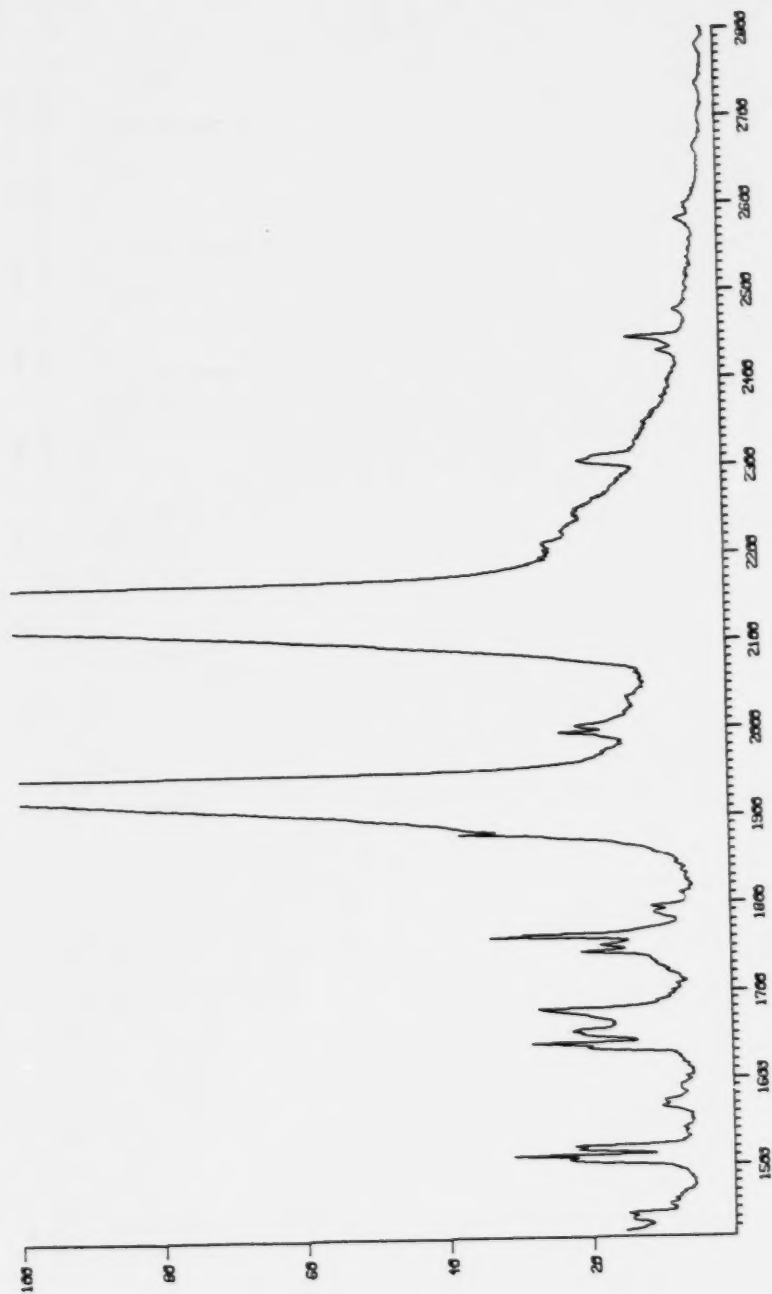


Fig. 4 (Cont'd). Total Ion Chromatogram of 8/3/76 H-1 Neutral Extract



Fig. 5. Total Ion Chromatogram of 8/3/76 H-1 Neutral Extract

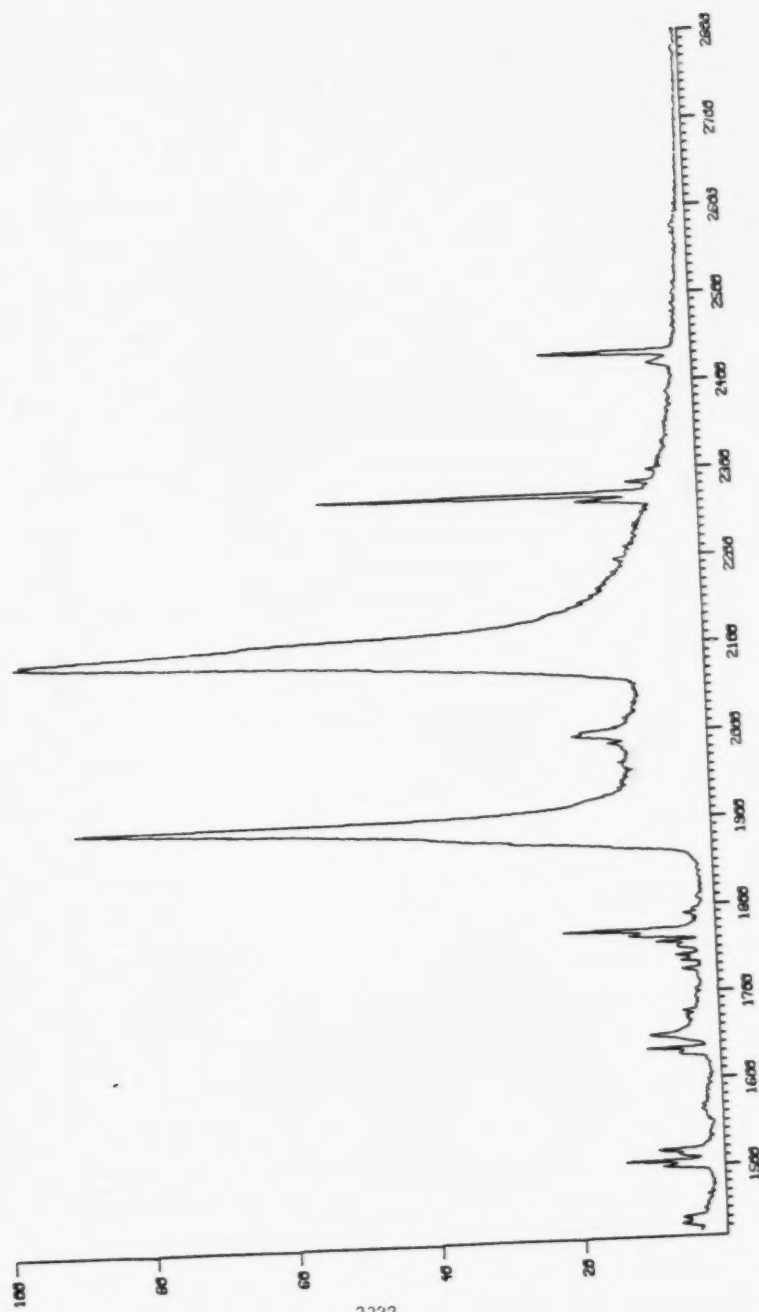


Fig. 5 (Cont'd). Total Ion Chromatogram of 8/3/76 H-1 Neutral Extract

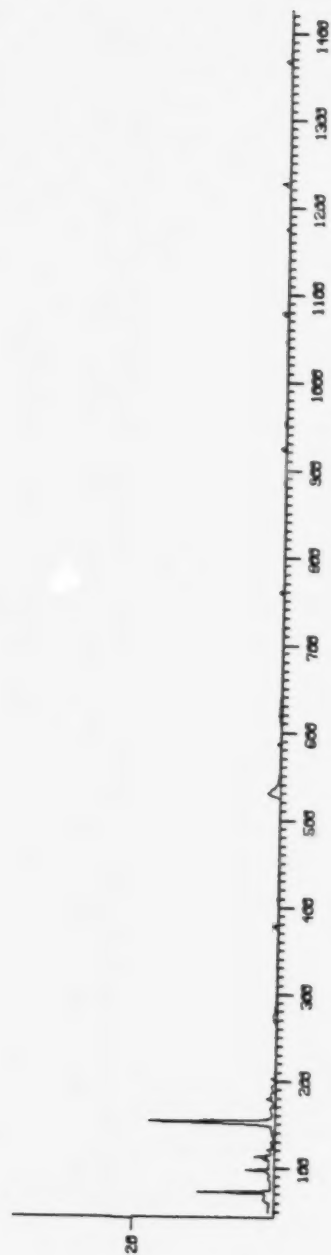


Fig. 6. Total Ion Chromatogram of 8/3/76 H-1 Base Extract

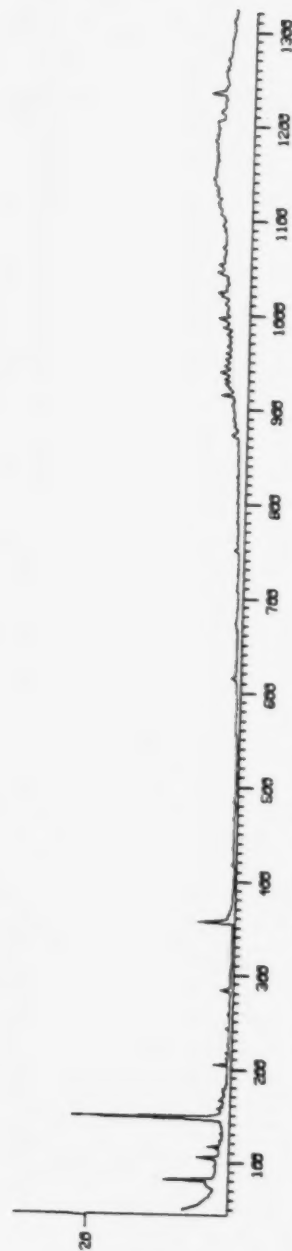


Fig. 7. Total Ion Chromatogram of 8/3/76 I-7 Neutral Extract

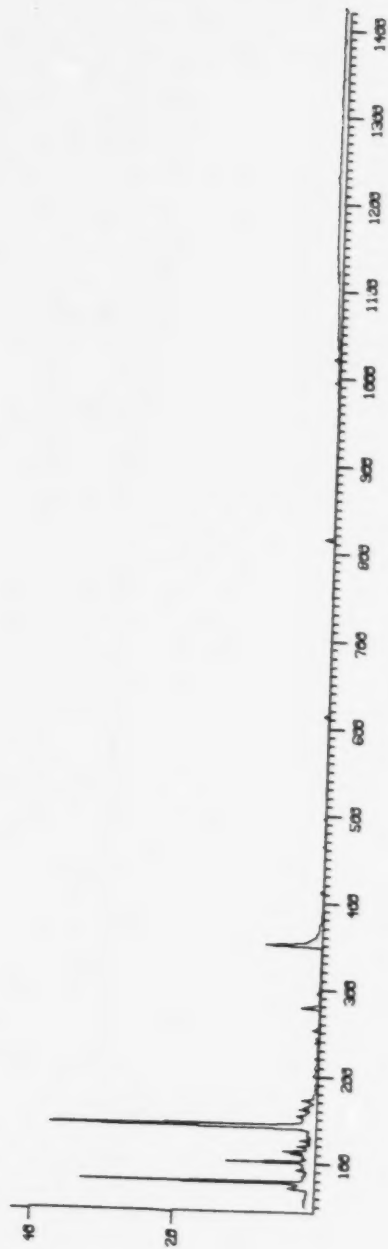


Fig. 8. Total Ion Chromatogram of 8/3/76 I-7 Acid Extract

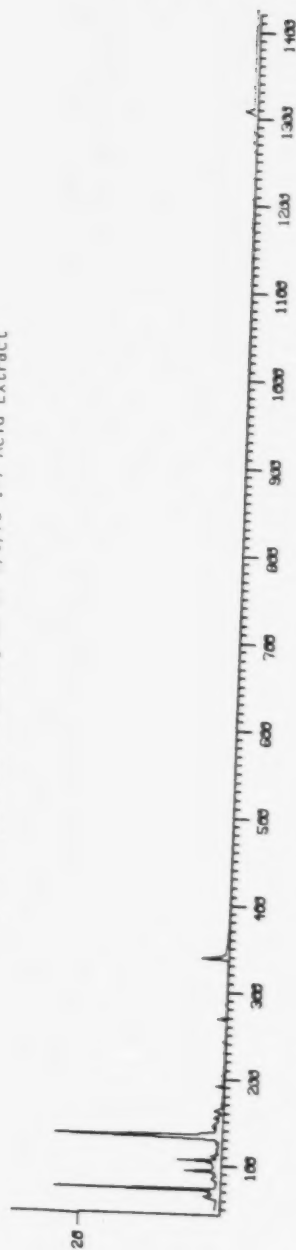


Fig. 9. Total Ion Chromatogram of 8/3/76 I-7 Base Extract



Fig. 10. Total Ion Chromatogram of 11/4/76 K-7 Neutral Extract



Fig. 11. Total Ion Chromatogram of 11/4/76 K-7 Acid Extract



Fig. 12. Total Ion Chromatogram of 11/4/76 K-7 Base Extract

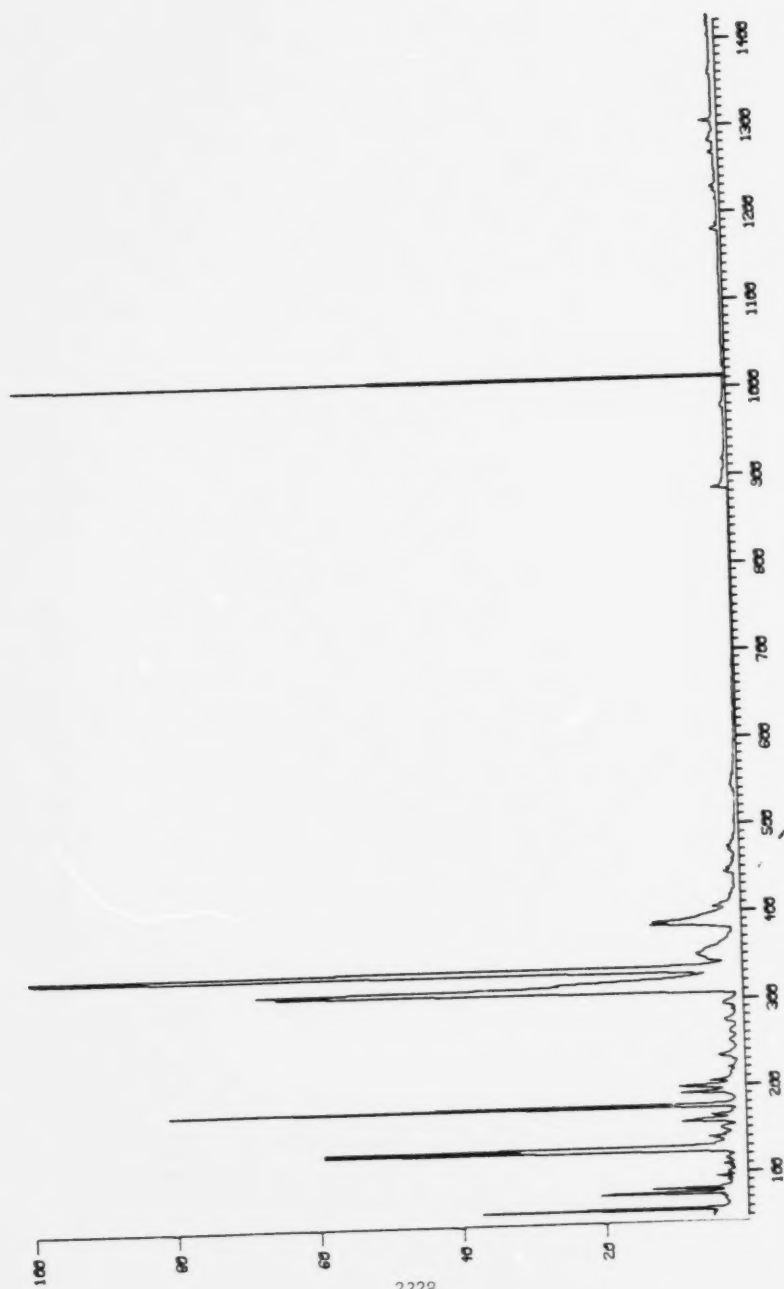


Fig. 13. Total Ion Chromatogram of 11/4/76 L-7 Neutral Extract

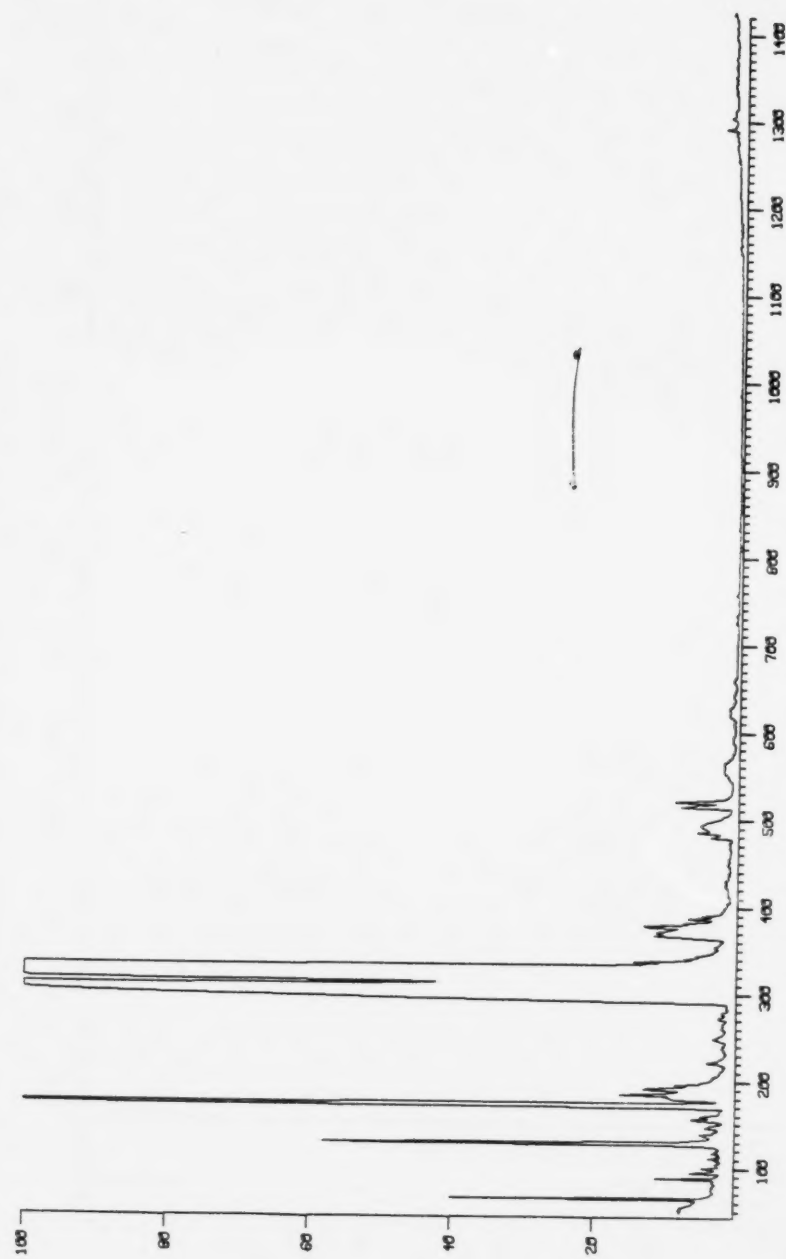


Fig. 14. Total Ion Chromatogram of 11/4/76 L-7 Acid Extract



Fig. 15. Total Ion Chromatogram of 11/4/76 L-7 Base Extract

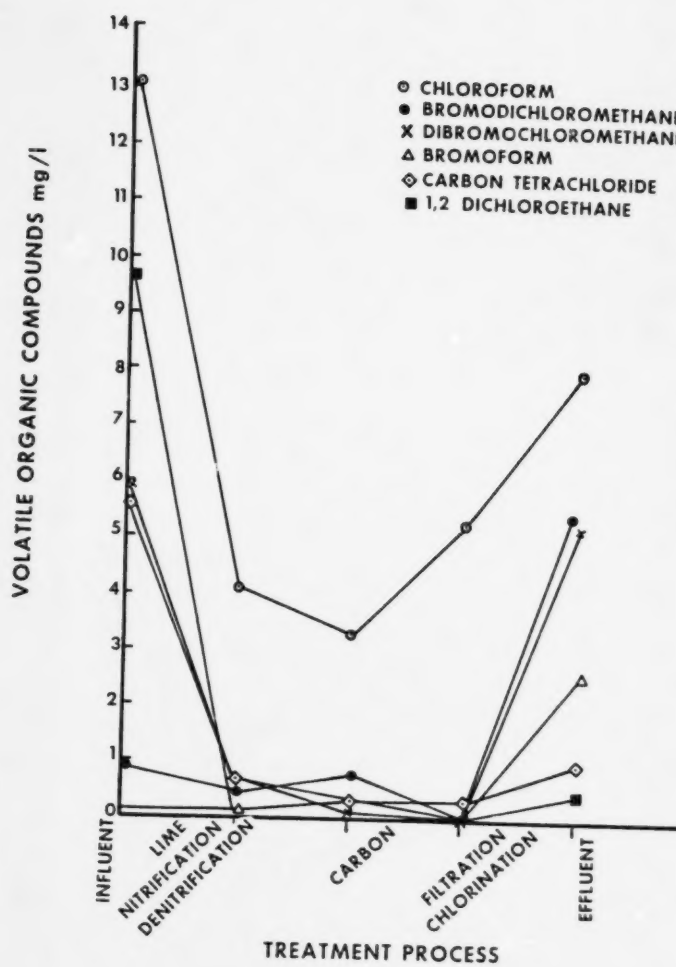


Fig. 16. Removal of Volatile Organics

BEHAVIOR OF VOLATILE AND EXTRACTABLE ORGANICS IN A COMBINED BIOLOGICAL / PHYSICAL / CHEMICAL TREATMENT OF A MUNICIPAL WASTE WATER

U.S. ENVIRONMENTAL PROTECTION AGENCY
MUNICIPAL ENVIRONMENTAL RESEARCH LABORATORY
WASTEWATER RESEARCH DIVISION
CINCINNATI, OHIO 45268

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